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DEVELOPMENT AND EVALUATION OF A SHOCK TUNNEL FACILITY FOR CONDUCTING FULL-SCALE TESTS OF LOADING, RESPONSE, AND DEBRIS CHARACTERISTICS OF STRUCTURAL ELEMENTS

December 1967

OCD Work Units 3313B, 1123D

Subcontract Nos. 11229(6300A-320), 11618(6300A-250)

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Summary
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OCD Work Units 3313B, 1123D

by

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Prepared for

Stanford Research Institute
Menlo Park, California

Subcontract Nos. 11229(6300A-320), 11618(6300A-250)

and

Office of Civil Defense
Office of the Secretary of the Army
Washington, D. C. 20310

This report has been reviewed in the Office of Civil Defense and approved for publication. Approval does not signify that the contents necessarily reflect the views and policies of the Office of Civil Defense.

Summary Report

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DEVELOPMENT AND EVALUATION OF A SHOCK TUNNEL FACILITY FOR CONDUCTING FULL-SCALE TESTS OF LOADING, RESPONSE, AND DEBRIS CHARACTERISTICS OF STRUCTURAL ELEMENTS

The objectives of this program were to convert an existing large-cross-sectional-area tunnel into a shock tube; to evaluate the capabilities of this facility for blast loading and response studies of full-scale and large-scale structural elements; and to design a detailed test program to investigate the loading, response and debris characteristics of wall panels.

The basic tunnel, which is a section of a former coastal defense complex, is rectangular in cross section and 163 ft long. The first 63 ft of the tunnel, which is used as the compression chamber, has an 8- by 8.5-ft cross section. The remaining portion of the tunnel, which expands in an 8-ft transition section to an 8.5- by 12-ft cross section, 92 ft long, is used as the expansion chamber. The tunnel conversion included blocking off several doorways and other openings along the side of the tunnel, installation of a cylindrical steel liner in the compression chamber, and installation of an instrumentation system. A sketch of the shock tunnel facility is presented in Fig. 1.

The tunnel is operated as a shock tube by means of the volume detonation technique, with primacord as the explosive material. In this mode of operation, the primacord is distributed uniformly throughout a section of the compression chamber portion of the tunnel. On detonation of the primacord, (which proceeds at a rate of about 20,000 ft/sec), a quasi-static pressure is built up very rapidly throughout the entire compression chamber. The expansion of this high-pressure gas into the remaining part of the tunnel generates the desired shock wave.

Two series of tests have been conducted to evaluate this facility for blast loading and response studies. These consisted of a program of air blast tests to check out the instrumentation system and calibrate the facility and a limited panel test series to demonstrate the capability of the shock tunnel and its associated equipment to handle and test full-scale wall panels.

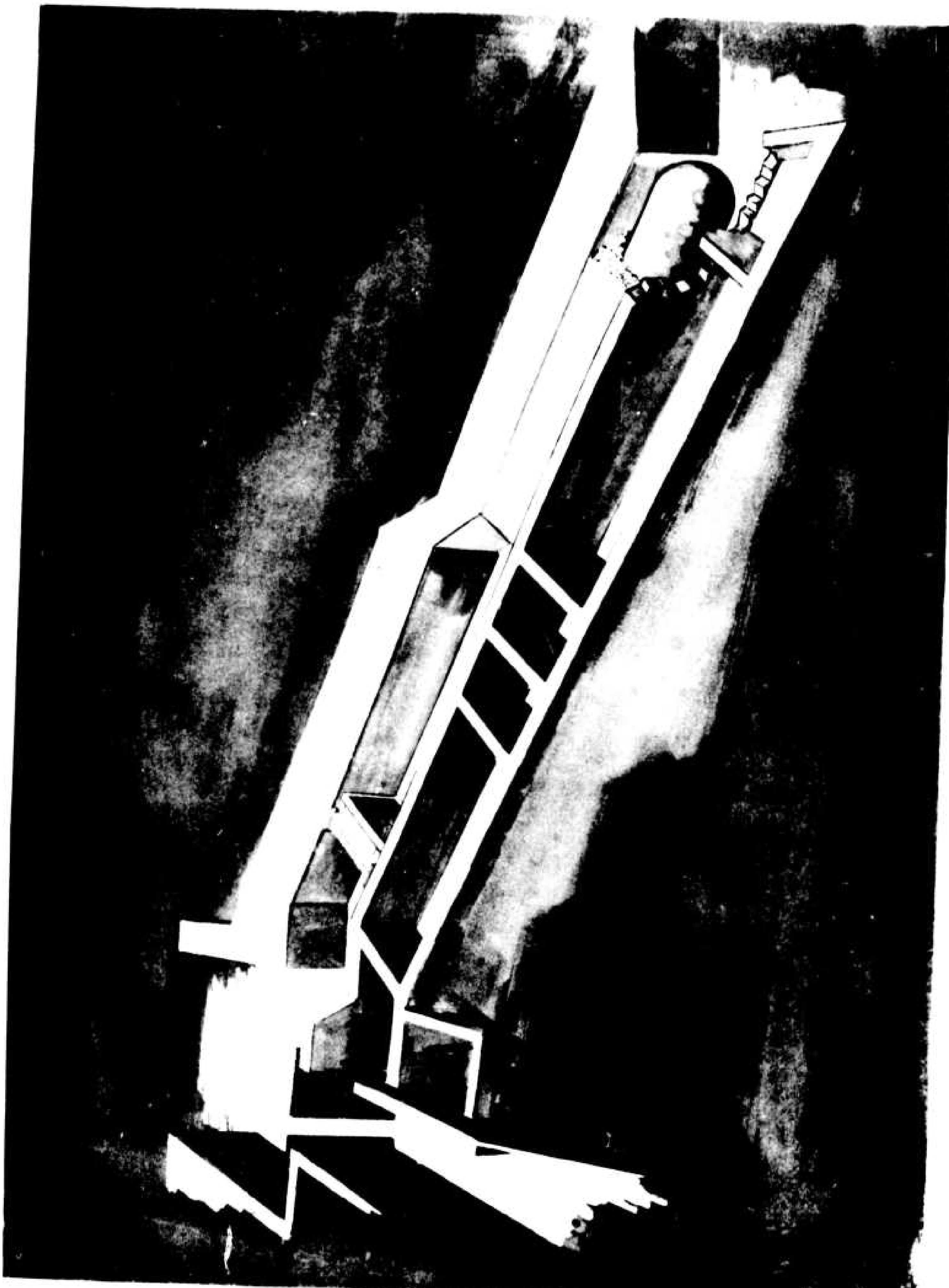

Fig. 1. URS Shock Tunnel Facility

In the air blast test series, approximately 50 tests were conducted in which the variables of charge density, charge location, charge length, method of detonation, and tunnel geometry (both open and closed) were investigated.

The charge densities used ranged from one strand of primacord, containing approximately 0.006 lb of explosive/ft, to eight strands of primacord, containing approximately 0.05 lb of explosive/ft.

The results of this series of air blast tests indicated that:

1. The pulse shapes and peak overpressures obtained from the same charge density and charge arrangement are reproducible.

2. Shock overpressures increased with increasing charge density. For example, the two-strand charges tests (approximately 0.0125 lb/ft), which varied in length from 40 to 60 ft, yielded a peak incident overpressure of 3.7 psi at the test section location, while similarly an eight-strand charge (approximately 0.05 lb/ft) yielded a peak incident overpressure of 9 psi. It is anticipated that pressures up to 12 psi (the maximum test value contemplated at present) can be obtained with no difficulty.

3. The pulse shape varies with charge arrangement. For example, when the primacord strands are placed together at the center of the tube, a classical shock wave is obtained. When the strands are separated and spread out in the tube, flat-topped shapes are obtained.

4. Pulse durations increase with increase in charge length, with typical durations for a 40-ft charge being approximately 40 to 60 msec and for a 60 ft charge 80 to 100 msec, for both the classical and flat-topped pulse shapes. The duration of the flat-topped portion of the latter pulses ranged from 30 to 50 msec.

The limited panel test series, which included seven failure tests of 8-in.-thick brick walls and three failure tests of 4-in.-thick timber stud walls, confirmed the suitability of the shock tunnel for full-scale panel testing. The wall panels were approximately 8.5 ft high 12 ft wide, were constructed in a steel frame and mounted to provide a simple beam support condition, i.e., the panels were pinned at the top and bottom and had no restraint at the sides.

The panels were held in the tunnel by wall blocks and plate girders which comprise a system for holding a panel in place in the tunnel and for making measurements of the total load imposed on the panel.

Five of the brick panels and two of the timber stud walls were subjected to peak incident pressures of ~1.5 psi, corresponding to a peak reflected pressure of ~3 psi. The remaining two brick panels and one timber stud wall were subjected to peak incident pressures of ~3.5 psi, corresponding to peak reflected pressures of ~8 psi.

The failure processes for these wall panels were reproducible, and the failure times, even for the brick panels, were much less than the loading durations.

As part of this program a detailed test plan to investigate the loading, response and debris formation and distribution.

The panel types studied will be selected to be representative of a class of panels whose general structural response characteristics are expected to be similar. It is anticipated that predictions for response and debris functions for other types of panels in the same class can be made on the basis of the improved theories without having to conduct additional extensive test series. Only a minimum amount of additional testing should be necessary to verify the applicability of the theory.

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ABSTRACT

This report describes the conversion of an existing large-cross-sectional-area tunnel into a shock tube and the evaluation of the capabilities of this facility for blast loading and response studies of full-scale and large-scale structural elements. Also included are the results from a preliminary test series of full-scale wall panels conducted as part of the evaluation program and the design of a detailed test program to investigate the loading, response, and debris characteristics of wall panels.

The basic tunnel, which is a section of a former coastal defense complex, is rectangular in cross section and 163 ft long. The first 63 ft of the tunnel, which is used as the compression chamber, has an 8- by 8.5-ft cross section. The remaining portion of the tunnel, which expands in an 8-ft transition section to an 8.5- by 12-ft cross section, 92 ft long, is used as the expansion chamber. The tunnel conversion included blocking off several doorways and other openings along the side of the tunnel, installation of a cylindrical steel liner in the compression chamber, and installation of an instrumentation system.

Shock waves are generated in the expansion chamber of the tunnel by detonating strands of primacord which have been uniformly distributed throughout a section of the compression chamber.

Evaluation tests conducted to date indicate that a wide range of air blast conditions can be generated depending on the explosive arrangement. Both peaked and flat-topped pulse shapes have been obtained, with total durations approaching 100 msec and flat-topped durations of about 40 msec. The upper overpressure operating limit is controlled at present by the strength of the expansion chamber and has been tentatively set at about 12 psi incident until further response information on the tunnel walls has been obtained.

The preliminary wall panel test series, which included seven 8-in.-thick nonreinforced brick panels and three timber stud walls, confirmed the suitability of the shock tunnel for full-scale panel testing. The failure process was reproducible, and the failure times, even for brick panels, were much less than the loading durations.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the assistance and guidance of Mr. James Halsey of Stanford Research Institute during the conduct of this program.

The authors also wish to acknowledge the contributions of URS staff members Messrs. H. Mason, J. Boyes, D. Walters and T. Bradfield.

FOREWORD

This interim technical report describes the progress on the shock tunnel development and evaluation program for the period November 1966 to October 1967. This work, which was performed under the direction of Stanford Research Institute for the Office of Civil Defense, was conducted under two separate work units. Phase I, which was funded under OCD Work Unit 3313B, was mainly concerned with determining the overall feasibility of using the tunnel for blast loading tests of structural elements and outlining an experimental program for such tests. Phase II, which was funded under OCD Work Unit 1123D, included additional development of the shock tunnel facility and preliminary tests of full-scale brick wall panels and timber stud walls. Because of the close interrelationship of the work performed on the two contracts, the results of each have been combined in this single report. The shock tunnel program is continuing under Work Unit 1123D, with primary emphasis on studies of loading, response, and debris characteristics of wall panels.

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INTRODUCTION

The overall objective of this program was to evaluate the capabilities of a large cross-sectional area tunnel for use as a shock tube in testing full-scale structural elements. The tunnel, which is a section of a former coastal defense complex, is 163 ft long and has a 8-1/2- by 12-ft cross section over the majority of its length. This study was undertaken after preliminary considerations suggested that it would be relatively simple and inexpensive to convert this tunnel into a shock tube and that its large rectangular cross section would make it uniquely suitable for a variety of blast loading and response studies.

SCOPE OF WORK

The program was conducted in two phases. Phase I, initiated in November 1966 under subcontract 11229(6300A-320), included the following tasks:

1. Converting the tunnel to a shock tube configuration.
2. Conducting a series of preliminary tests to determine the range of air blast characteristics obtainable.
3. Establishing and proving instrumentation.
4. Designing a detailed test program to investigate loading, response, and debris characteristics of panels.

The results of the initial series of tests, which were presented in a progress report submitted in February 1967, were sufficiently encouraging that Phase II of the program was intiated in April 1967 under subcontract 11618(6300A-250), this phase included the following tasks:

1. Tunnel tests to select the explosive charge arrangement and panel location for initial panel tests.
2. Additional facility improvements necessary for fabrication and handling of wall panels and for repetitive testing.
3. Review of existing information on the structural response of wall panels.
4. Tunnel tests of selected brick panel types and one type of timber stud wall.

REPORT ORGANIZATION

This report is divided into two parts: Part I covers the development of the basic shock tunnel facility; Part II deals with the panel test program.

Part I includes a description of the original facility and of the numerous modifications and improvements which have been made to it; a discussion of the operating concepts; a presentation of the data from the air blast tests which were conducted to calibrate the facility and determine optimum charge arrangements, and a general discussion of the types of structural element testing which appear practical with the loading capabilities developed to date.

Part II includes the results of the limited panel test series conducted during this program, and a recommended comprehensive test program to investigate loading, response, and debris characteristics of wall panels.

PART I

Section 1
SHOCK TUNNEL DESCRIPTION

The Shock Tunnel Facility is located at the URS Physical and Engineering Sciences Field Laboratory in the San Francisco Bay Area near the north end of the Golden Gate Bridge. This laboratory is underground, in a former coastal defense gun emplacement complex, and contains approximately 23,000 sq ft of floor space, which is divided into shops, instrumentation rooms, a wave tank, explosive magazines, and the shock tunnel facility. A cutaway view of this laboratory is shown in Fig. 1.

The Shock Tunnel Facility occupies approximately 8,000 sq ft of the laboratory. The shock tunnel, shown in Figs. 2 and 3, is rectangular in cross section, 163 ft long, and has walls of reinforced concrete varying from 3 to 12 ft in thickness. The roof is also reinforced concrete, 6 to 16 ft thick. The first 63 ft of the tunnel which is used as the compression chamber has an 8- by 8.5-ft cross section. The remaining tunnel then expands in an 8-ft transition section to an 8.5- by 12-ft cross section, 92 ft long (the 12-ft dimension being horizontal). This portion of the tunnel is used as the expansion chamber.

There are eight side openings into the basic tunnel; five 6-ft-wide, 7-ft-high doorways, which leads to the two powder rooms and to the storeroom; two 15-ft-wide, 8.5-ft-high openings to the shell rooms; and a 16-ft-wide, 13-ft-high opening at the rear of the casemate. The front or west side of the casemate is also open and serves as the exhaust port for the tunnel.

Fig. 1. Cutaway View of the URS Physical and Engineering Sciences Field Laboratory

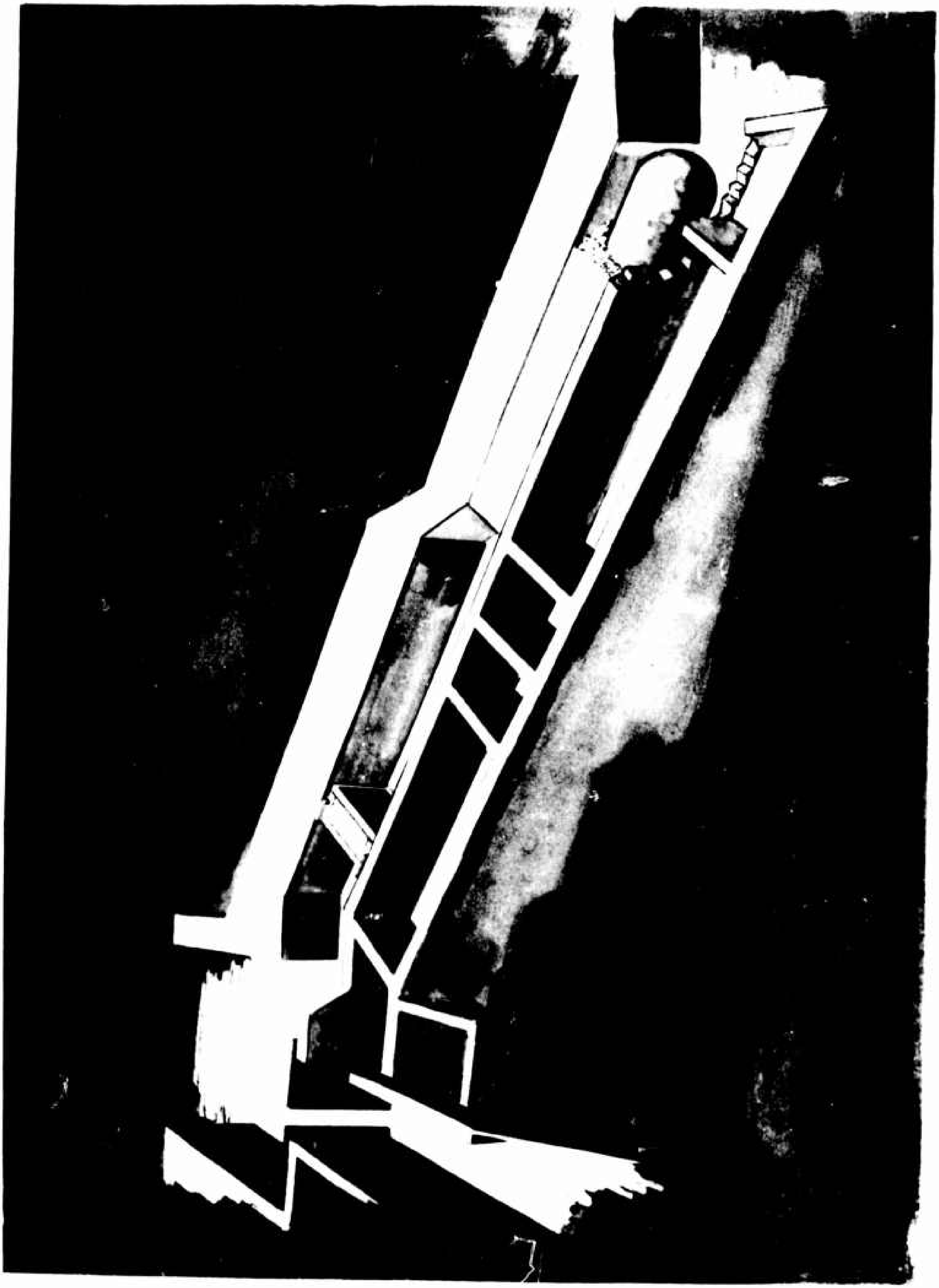


Fig. 2. Cutaway View of Shock Tunnel Facility

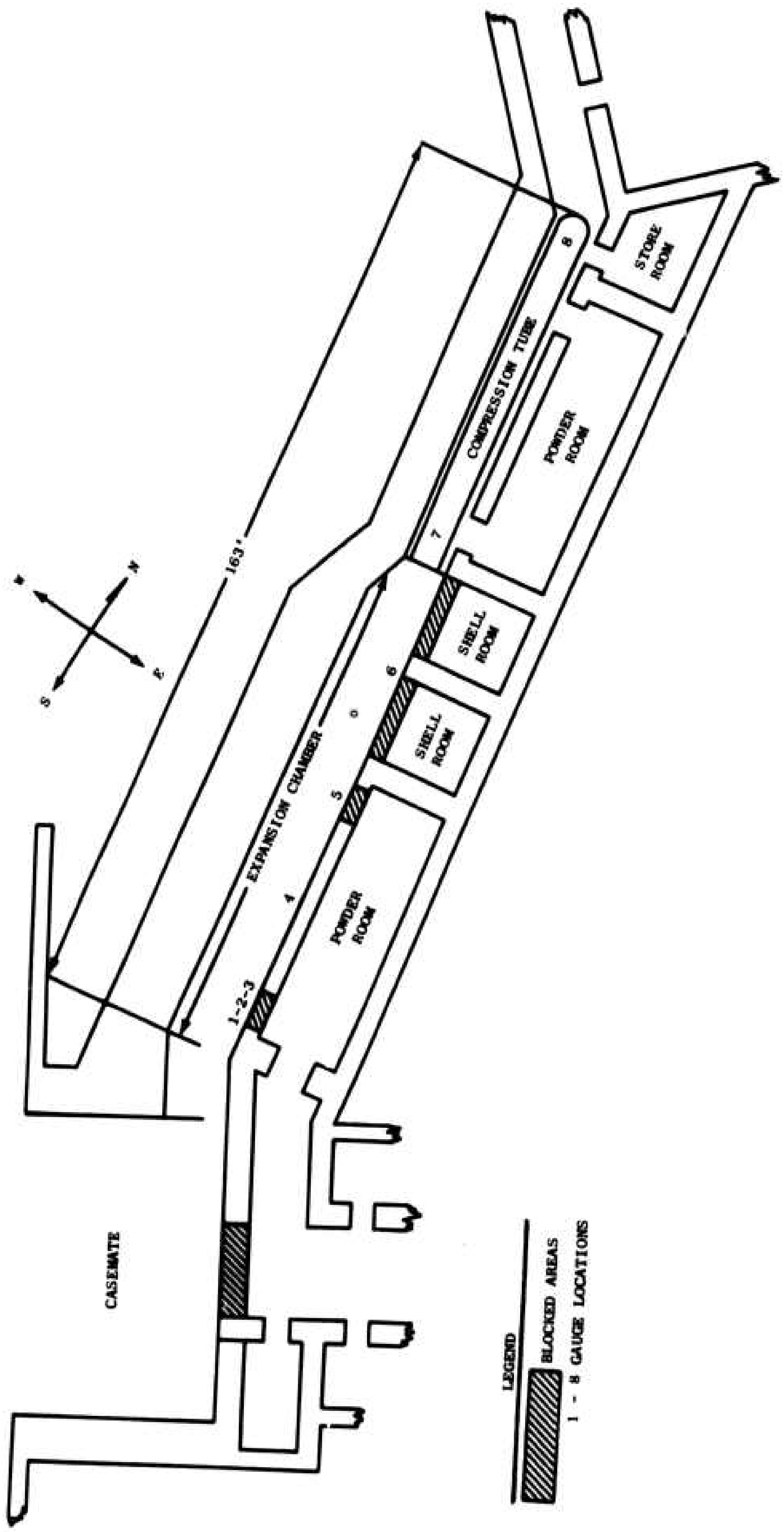


Fig. 3. Plan View of Shock Tunnel Facility

Section 2
MODIFICATIONS AND IMPROVEMENTS

To convert the existing tunnel into a shock tube required the following modifications:

- Design and installation of the compression tube.
- Installation of closures along east wall of tunnel.
- Installation of an instrumentation system.

THE COMPRESSION CHAMBER

Preliminary calculations made at the start of the program had indicated that peak incident shock overpressures of ~ 12 psi in the expansion chamber would be sufficient as an upper limit for the majority of tests on structural building panels. Based on work by previous investigators (Ref. 1) a peak incident shock overpressure of 12 psi would then require approximately 50 psi in the compression chamber.

It was originally planned to use the compression chamber section of the tunnel essentially as it was, with the only modifications being the provision of an end closure and several side closures for doorways which opened onto this section of the tunnel and the reinforcement of the 3-ft-thick wall along the one side of the tunnel.

It soon became evident, however, that the amount of work required to anchor the end closure and to reinforce the 3-ft-thick wall for the design load of 50 psi would be quite extensive and that the simplest and most economical solution would be to install a steel liner in the compression chamber section of the tunnel. Accordingly, a 3/8-in.-thick, 7-ft 8-in.-diameter steel tube, 63 ft long with a 1/2-in.-thick domed end closure, was installed.

For the first calibration tests the compression chamber tube was fastened in place by ceiling brackets, which were held in place by both epoxy cement and bolts (left after removal of the shell handling track) and by heavy steel channels, which were welded to the side of the tube and fit snugly into the two

powder room doors on the east side of the tube. To prevent collapse of the compression tube on rebound, 12 screw jacks were installed in the tube. The jacks were installed in pairs, 90 degrees apart, and were spaced every 10 ft down the tube. The ends of the screw jacks pushed against 4- by 6-in. timbers running the full length of the tube. The space around the mouth of the tube-between the tube shell and the concrete walls, floor, and ceiling—was sealed with sandbags.

This method of fastening and sealing of the compression tube was sufficient for the proof-testing and calibration series but was not considered suitable for a continuing panel test series. The sandbag seal around the tube was largely destroyed during each test, was a source of dust, and took considerable time and manpower to replace. In addition, there was some concern that the ceiling brackets and steel channels might not be sufficient to hold the tube in place at the higher pressure levels required for some of the panel tests.

Accordingly, an investigation was conducted into various methods of restraining the tube, including a massive steel frame around the mouth of the tube combined with stringers and rings welded on the inside of the tube and a somewhat novel approach of filling the space between the tube shell and the concrete walls of the tunnel with foamed-in-place rigid urethane foam. This latter method was very appealing in that it would provide a continuous elastic support around and along the entire tube to prevent tube collapse. Also it would supply the necessary hold-down strength and a large amount of damping on the tube to facilitate making pressure measurements on the walls of the tube. As a further side benefit, it would reduce noise and vibration throughout the facility. However, since this would be a new application for this material, it was necessary to run a series of tests to determine if the foam would bond to the tube shell and to the concrete wall and if the foam would expand in the cold atmosphere of the shock tunnel facility. It had been determined from preliminary calculations that a bond strength of 10 psi tensile and 10 psi shear would be required.

A series of tests were conducted in which samples of foam were poured on sections of the tube shell and tunnel wall. Various cleaning procedures were used on the shell and tunnel wall, and tests were conducted for a range of wall and air temperatures.

These samples were tested for shear bond and tensile bond strength, and it was determined that if the tunnel and shell were washed with water and the tunnel and tube warmed to at least 70 degrees, shear bond strengths of 25-28 psi and tensile bond strengths of 28-35 psi could be obtained.

In addition it was determined that the cost of the foam would be about the same as the massive steel frame but that the foam would have the advantages discussed above.

A series of small holes were cut in the tube shell to facilitate placement of the foam, the tube and tunnel walls were washed with a fire hose, and a series of heaters were placed in the tube shell. At an ambient temperature of 70 to 80 degrees in the tunnel, approximately 1200 lb of urethane foam were pumped through the holes in the tube shell. The resulting in-place density of the foam is approximately 2 lb/cu ft. A thin steel collar was installed around the mouth of the tube to protect the foam from air blast.

One additional improvement was the addition of a 24-in.-diameter access way with a blast door, which was installed in the dome end of the tube. The purpose of this door is to allow access to the compression chamber for installing the explosive charges after the tunnel is blocked with a test specimen, and to provide ventilation in the tunnel.

INSTALLATION OF CLOSURES

The installation of the compression tube eliminated three openings in the tunnel. The five remaining openings in the east wall, the two shell room openings, and two doorways to the powder rooms and the opening to the casemate were closed by timber barricades. The two shell room openings and the powder room doors were provided with a front wall of 16-in.-thick timbers backed up by a bracing network of 12-in. by 12-in. and 8-in. by 12-in. timbers extending to the far wall of the powder and shell rooms. The casemate closure was constructed from 12-in. by 12-in. timbers set in a groove in the concrete floor and braced at the top by a steel frame. A 5-ft section of the closure was made into a sliding door to allow ready access to the casemate and tunnel areas from the panel storage area.

SHOCK TUNNEL PRESSURE INSTRUMENTATION SYSTEM

The pressure instrumentation system consisted of quartz piezoelectric transducers, charge amplifier preamps, and dual-beam oscilloscopes equipped with Polaroid cameras. The eight gauge locations used are shown in Fig. 3. The gauges at locations 1 through 6 were positioned midway between the floor and ceiling in the east wall. The gauges at locations 7 and 8 were installed in the side of the tube. The approximate gauge distances measured from the dome end of the compression chamber are given below. (The test panels are positioned at approximately 146 ft from the dome end.)

Location	Distance (ft)	Parameter Measured
1	146	overpressure
2	146	overpressure
3	146	overpressure
4	127	overpressure
5	104	overpressure
6	85	overpressure
7	57	overpressure
8	17	overpressure

STRUCTURAL ANALYSIS OF THE EXPANSION CHAMBER

A structural analysis of the expansion chamber area of the tunnel was conducted to point up any critical facility weaknesses and to establish the highest safe operational level for the shock tube.

This analysis, indicated that the major problem area in the tunnel was the 3-ft-thick east wall, and that a safe upper limit for this wall, without structural modification, would be somewhat in excess of 26 psi. It should be noted, however, that a great deal of information about the facility, such as concrete strength, soil strength, construction procedure, etc., was not known, and this may be a relatively conservative estimate.

It is planned to make Stresscoat, stress gauge, and other such measurements on the facility during the test program in an effort to obtain further information

required to make a more refined analysis. Also, it is planned to make frequent and careful inspections of the facility to watch for any change in behavior which could be indicative of a fatigue type of failure, since little is known about the response of concrete to repetitive shock loadings.

Section 3
OPERATING CONCEPT

The tunnel is operated as a shock tube by means of the volume detonation technique, with primacord as the explosive material. In this mode of operation, the primacord is distributed uniformly throughout a section of the compression chamber portion of the tunnel. On detonation of the primacord (which proceeds at a rate of about 20,000 ft/sec), a quasi-static pressure is built up very rapidly throughout the entire compression chamber. The expansion of this high-pressure gas into the remaining part of the tunnel generates the desired shock wave.

Unlike conventional compressed-gas shock tubes, it is not necessary to separate the compression chamber from the expansion chamber with a frangible diaphragm. The detonation of the primacord is sufficiently rapid that the pressure buildup in the compression chamber is affected very little by the small amount of expansion of the overall volume during the buildup process. The primacord is strung on pipe "X" bracing installed in the compression chamber as shown in Fig. 4.

Previous work with the volume detonation technique (Ref. 1 and 2) has indicated that, qualitatively, the shock behavior is similar to that in conventional shock tubes. After detonation, the shock generated in the expansion chamber propagates with a constant peak pressure until the rarefaction proceeds to the end of the compression chamber, reflects off the end, and finally overtakes the shock front. Prior to this time, the shock pulse would have a flat top. After the rarefaction overtakes the shock front the pulse is peaked and attenuates with increasing distance. The actual overtaking distance increases with increasing charge length. For a given charge length, the total pulse duration increases with increasing distance down the tunnel, but the flat-topped duration decreases.

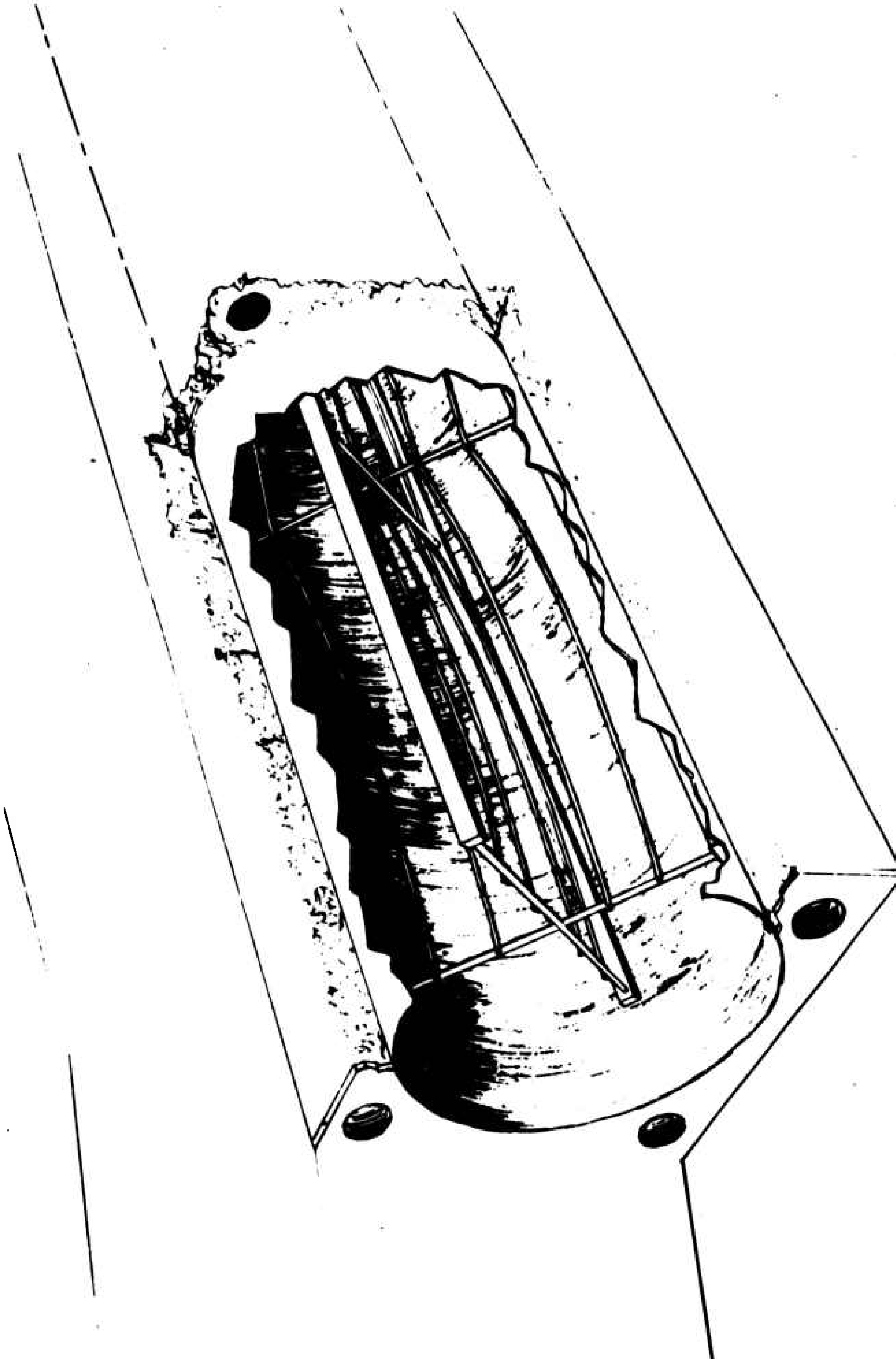


Fig. 4. Primacord Array

Section 4
AIR BLAST TESTS

A continuing series of air blast tests have been conducted throughout the program. These tests have been conducted in part to check out the air blast instrumentation system and in part to calibrate the shock tunnel facility. Approximately 50 tests were conducted in which the variables: charge density, charge location, charge length, and method of detonation and tunnel geometry (both open and closed) were investigated.

The charge density used ranged from one stand of primacord, containing approximately 0.006 lb of explosive/ft, to eight strands of primacord, containing approximately 0.05 lb of explosive/ft. The primacord strands were mounted longitudinally in the compression chamber on a pipe "X" frame. In most of the tests, the primacord was initiated at the closed end of the compression chamber. However, three tests were conducted with initiation at the open end and one with initiation at the middle of the charge. Tests were also conducted with the tunnel open and in a few cases with the mouth of the tunnel closed with a timber barricade.

A listing of the majority of these tests is presented in Table 1. Included in this table are: the test number (which includes the date of the test); the length and number of strands in the explosive charge; the location of the primacord strands in the compression chamber (expressed in radial distance from the axis of the tube, i.e., fraction of a radius from the tube axis); the detonation location OE, detonated near the open end of the compression tube; CE, detonated near the closed end of the compression tube, and MC, detonated in the middle of the charge; the tunnel geometry, either open or closed; the maximum peak incident overpressure; and the positive-phase duration.

The pulse shapes obtained included: a classical pulse, i.e., fast rise followed by an exponential decay and a more common flat topped pulse. This latter pulse typically has a fast rise followed by a generally flat-topped region and then a nearly linear decay. Typical pulse shapes are shown in Fig. 5.

The results from this series of air blast tests indicate that:

Table 1

AIR BLAST TESTS

TEST NUMBER	EXPLOSIVE CHARGE			DETONATOR LOCATION	TUNNEL GEOMETRY	MAXIMUM PEAK OVERPRESSURE	DURATION
	LENGTH	NUMBER OF STRANDS	LOCATION				
02-16-67-1	30	2-40'	1/2	CE	Open	-	-
02-16-67-2	30	2-40'	1/2	CE	Open	~2	38
02-17-67-3	30	4-40'	1/2	CE	Open	2.5	40
02-17-67-4	40	4-40'	1/2	CE	Open	3.7	56
02-20-67-1	40	4-40'	1/2	CE	Open	-	-
02-20-67-2	40	8-40'	1/3 & 2/3	CE	Open	8	55
02-21-67-1	40	4-40'	1/2	CE	Open	3.5	50
02-21-67-2	40	8-40'	1/3 & 2/3	CE	Open	9	58
02-23-67-1	40	4-40'	1/2	CE	Open	3.0	52
02-23-67-2	40	4-40'	1/2	OE	Open	3.4	56
02-23-67-3	40	4-40'	1/2	OE	Open	3.4	58
02-23-67-4	60	4-60'	1/2	OE	Open	3.7	80
03-13-67-1	40	4-40'	1/2	OE	Open	3.7	-
03-13-67-2	40	2-40'	1/2	OE	Open	1.7	60
03-13-67-3	40	4-40'	1/2	OE	Open	3.7	78
03-14-67-1	40	1-160'	1/2	CE	Open	3.5	58
03-14-67-2	40	1-160'	Spiral	CE	Open	4	55
03-14-67-3	40	4-40'	1/2	M	Open	3.9	56
03-14-67-4	40	4-40'	0	CE	Open	5.6	60
03-14-67-5	60	1-240'	3/4	CE	Open	3.0	82
03-14-67-6	60	1-300'	0 & 3/4	CE	Open	4.8	80
03-14-67-7	78	1-390'	0 & 3/4	CE	Open	5	90
03-21-67-1	60	4-60'	3/4	CE	Open	3.7	58
03-21-67-2	60	4-60'	3/4	CE - Delay 1	Open	2.0	84
03-21-67-3	60	5-60'	0 & 3/4	CE - Delay 2	Open	2.5	78
08-02-67-1	60	1-60'	0	CE	Open	-	-
08-02-67-2	53	1-53'	0	CE	Open	1.1	80
08-02-67-3	60	1-60'	0	CE	Open	1.2	87
08-02-67-4	60	2-60'	1/2	CE - Delay 1	Open	2.2	95
09-25-67-1	60	4-60'*	3/4	CE	Closed	-	-
09-25-67-2	60	1-60'	0	CE	Closed	0.9	90
09-25-67-3	60	4-60'*	3/4	CE	Closed	1.1	100
09-25-67-4	60	4-60'	3/4	CE	Closed	3.25(6.4R)	90
09-25-67-5	60	4-60'	3/4	CE - Delay 2	Closed	2.7 (6.3R)	93

* Half-strength primacord.

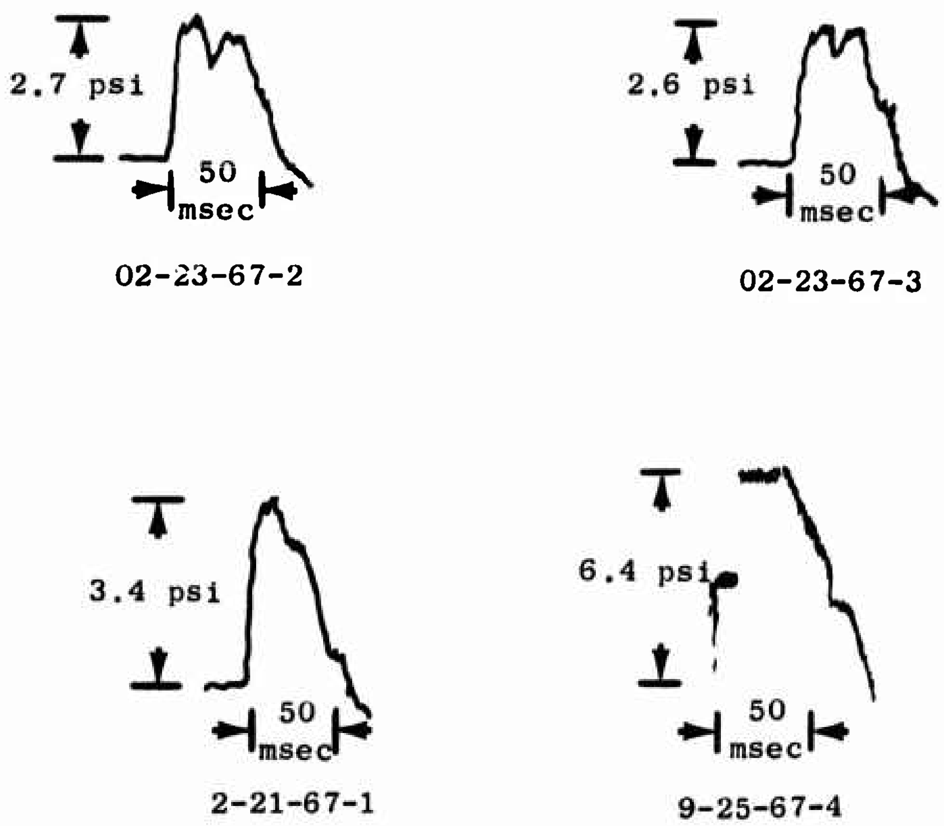


Fig. 5. Typical Air Pressure Gauge Traces (for test conditions see Table 1)

1. The pulse shapes and peak overpressures obtained from the same charge density and charge arrangement are reproducible.

2. Shock overpressures increase with increasing charge density. For example, the two strand charges tested (approximately 0.0125 lb/ft) which varied in length from 40 to 60 ft, yielded a peak incident overpressure of 3.7 psi at the test section location while similarly an eight strand charge (approximately 0.05 lb/ft) yielded a peak incident overpressure of 9 psi. It is anticipated that pressures up to 12 psi (the maximum test value contemplated at present) can be obtained with no difficulty.

3. The pulse shape varies with charge arrangement. For example, when the primacord strands are placed together at the center of the tube, a classical shock wave is obtained. When the strands are separated and spread out in the tube the flat-topped shapes are obtained.

4. Pulse durations increase with increase in charge length with typical durations for a 40 ft charge being approximately 40 to 60 msec and for a 60 ft charge 80 to 100 msec, for both the classical and flat-topped pulse shapes. The duration of the flat-topped portion of the latter pulses ranged from 30 to 50 msec.

It should be emphasized, however, that only a limited number of test conditions have been covered in the air blast tests conducted to date. The primary emphasis in this program was setting up and calibrating the facility for testing structural wall panels. Since the pulse shapes obtained to date seemed suitable for failure tests of the majority of wall panels of OCD interest it was determined that further exploratory air blast testing was not warranted during this program.

There are, however, indications that the flat-topped portion of the pulses and the total duration of the pulses can be lengthened considerably by the use of different weights of primacord, changes in the charge arrangement or placement in the compression tube, and by the use of sequenced detonations.

Section 5

GENERAL DISCUSSION OF THE APPLICABILITY OF THE SHOCK TUNNEL FOR BLAST LOADING AND RESPONSE STUDIES

One of the major objectives of the work to date has been to determine the capabilities of the shock tunnel for blast loading and response testing of structural elements. The use of the tunnel for testing wall panels has been particularly emphasized, and the capabilities for this purpose are discussed in detail in Section 2 of Part II as part of the design of the detailed wall panel program. The discussion presented in this section summarizes the capabilities for a broader range of structural elements.

The three general types of loading and response studies which it appears possible to conduct in the shock tunnel are:

1. Full-scale simulation tests - Tests in which the actual full-scale loading environments produced by nuclear weapons are simulated.

2. Large-scale model tests - Tests in which the shock wave is considered a scaled-down version of a full-scale blast wave.

3. Response mechanism tests - Tests in which the basic purpose is to study the response mechanism of objects to the general types of dynamic loadings produced by blast waves (with no attempt at simulating any particular wave). Such studies would be used to help in the development and verification of response theories which would be used to predict actual results for any given full-scale blast wave.

The suitability of the shock tunnel for conducting each of these general types of studies is described below. The limitations on the sizes of objects that can be tested in the shock tunnel are common to all three types of studies and are covered first, followed by specific remarks about each type of testing.

TEST OBJECT SIZE LIMITATION

For any type of testing in the shock tunnel, one restriction, of course, is that the objects fit within the test section of the tunnel (8-1/2 by 12 ft in cross section) and that by their presence they do not significantly disturb

the blast wave. The size of the objects that can be tested without disturbance depends in part on the method of mounting and exposing the objects. Wall panels, for example, can be exposed either face-on or side-on. In the face-on orientations, the panel and its mounting frame would completely block the tube so that the panel would be subjected to the reflected pressure pulse. In this case the test panels could approach the size of the tunnel cross section, since the entire section has to be blocked off in any case. Other objects which can be suitably mounted in the test frame and still provide essentially complete blockage of the tunnel can also be exposed in the face-on mode to a single loading pulse.

Wall panels and other objects can also be tested in the side-on orientation by mounting them in the side of the tunnel. Here again the vertical dimension of the object can approach the height of the tunnel. With little modification, widths up to about 6 ft can be handled by using an existing doorway; and by removing a section of the wall, greater widths can be handled. A 15-ft opening is available in the shell room about 19 ft past the transition section and can be used if wave shape requirements are not critical.

The biggest size limitation is on objects which it is desired to subject to all-around loading. Ideally, for this mode of loading, the objects' cross-sectional area presented to the flow should be small compared to the total cross section. In many cases, however, it is possible to test objects having cross-sectional areas of from 20 to 25 percent of the tube cross section (or 20 to 25 sq ft) without too much disturbance (Ref. 3).

FULL-SCALE SIMULATION TESTS

The flat-topped portion of the shock wave closely approximates the initial portion of the blast wave from a megaton-range weapon. Thus a close simulation of actual field loading and response conditions can be achieved in the shock tunnel for the following two general classes of target and loading conditions:

1. Targets whose natural periods are such that the times to maximum deflection (including failure) are less than the duration of the flat-topped portion of the wave.

2. Targets whose effective loading duration in a nuclear blast environment would be limited by the clearing times of the reflected pressure wave rather than by the actual free field positive-phase duration. Examples of such targets could be structural elements on the front faces of structures. (Typical front-face clearing times for structures with a minimum dimension (height or half-width) of 30 ft would be about 65 msec.)

It should be noted that wall panels are one of the most important classes of targets which appear to fit within one or both of these conditions.

Overpressure levels as high as 25-30 psi appear practical for objects that can be tested in the face-on mode. These pressures are adequate to cause failure in most of the common building structural elements of concern.

For side-on loading, the maximum pressures are on the order of 10 to 12 psi. This is sufficient to cause significant response of most elements of interest, but obviously not to the point of failure in all cases.

For targets and loading conditions which do not satisfy either of the above two conditions, the use of the shock tunnel as a direct full-scale simulant is somewhat limited, since the expected maximum durations are less than those from a 1-kt weapon.

SCALE-MODEL TESTS

The general concepts discussed for the full-scale simulation tests also apply for scale-model tests; however, the pulse duration limitations are of even less concern. The natural period of the scale-model objects and, thus, the time to maximum deflection would be significantly shorter, so that even a larger number of target-loading conditions would be satisfied by one of the two criteria discussed earlier. Even when these criteria are not satisfied, the total pulse duration would correspond to that from a much larger weapon.

RESPONSE-MECHANISM TESTS

The basic purpose of this type of testing is to improve our understanding of the response mechanism of various objects to the type of dynamic loading produced by blast waves. It is not necessary to be as specific about the loading

requirements for this type of testing, since if the response mechanisms of a given element or system can be understood, for a given loading, then this understanding can be extended to the wider range of more complex loadings of a nuclear weapon environment. It is clear, however, that qualitatively, there are certain test capabilities which are necessary or at least very desirable.

One of the most important of these is the ability to handle full-scale, or at least large-scale, test objects. The response of most objects to dynamic loading is quite complicated, and there is always some uncertainty in working with a scale model as to whether the mode of response has been properly scaled; in fact, it almost requires knowledge of the behavior being investigated to be sure that such is the case. The degree of uncertainty, however, decreases the closer the model approaches the full scale. The shock tunnel having such a large rectangular cross-sectional area well satisfies this requirement.

A second important capability is to be able to provide the general type of dynamic loading produced by the blast waves from nuclear weapons, i.e., load pulses having a rapid rise to a peak value with a sufficient duration to cause significant response in the test objects. It is also necessary to provide controlled variation in magnitude and duration of loading in order to properly evaluate response theories, and in at least some case, the loading should be sufficient to cause failure.

Since the shock tunnel loading for some cases can actually simulate the full-scale blast wave and is expected to cause failure in structural elements of concern such as wall panels, it clearly well satisfies the general type and magnitude of loading requirements. Rather wide variations in loading conditions are also possible by using various combinations of charge length, charge density, charge arrangement, firing sequence, and test position.

PART II
PANEL PROGRAM

INTRODUCTION

A discussion of the panel test program is presented in this part of the report. Section 1 is concerned with the hardware, instrumentation, panel construction, and results from a series of preliminary panel tests conducted during the performance of this program. Section 2 covers the design of a comprehensive program to investigate the loading, response, and debris characteristics of full-scale wall panels.

Section 1
PANEL TEST SERIES

The panel test series was conducted to demonstrate the capability of the shock tunnel and its associated equipment to handle and test full-scale wall panels. The series consisted of seven failure tests of 8-in.-thick brick walls and three failure tests of 4-in.-thick timber stud walls.

A simple beam support condition was used for this series i.e., the panels were pinned at the top and bottom and had no restraint at the sides. The rationale behind this particular support system is presented in the next section.

PANEL CONSTRUCTION

The panels for this test series were approximately 8.5 ft high and 12 ft wide. They were constructed in a steel frame which was designed to hold, support, and protect the panels during construction, storage, transporting and testing. Provided on this frame was the hardware required to create the edge connections necessary for the particular panel and test condition, lifting eyes for transport of the panel, and the hardware necessary for fastening the panel into the test section of the tunnel.

In the design of this frame, the critical part is the bottom member, since it must support the weight of the panel and transmit this load to the lifting rings. The design criterion here required locating the lifting rings to minimize the deflection and to keep the deflection within the limits one might expect in a building frame. The resulting frame has a deflection-to-span ratio of 1/440, which is less than the typical 1/360 used in building construction. It may be argued that deflection during construction is of little importance, since the mortar is a viscous fluid and would conform to the support shape. However, since it takes time to build the wall and for the mortar to "cure," it was felt that by adhering to ordinary building standards, support deformation would be avoided. A sketch of a brick panel and frame is presented in Fig. 6.

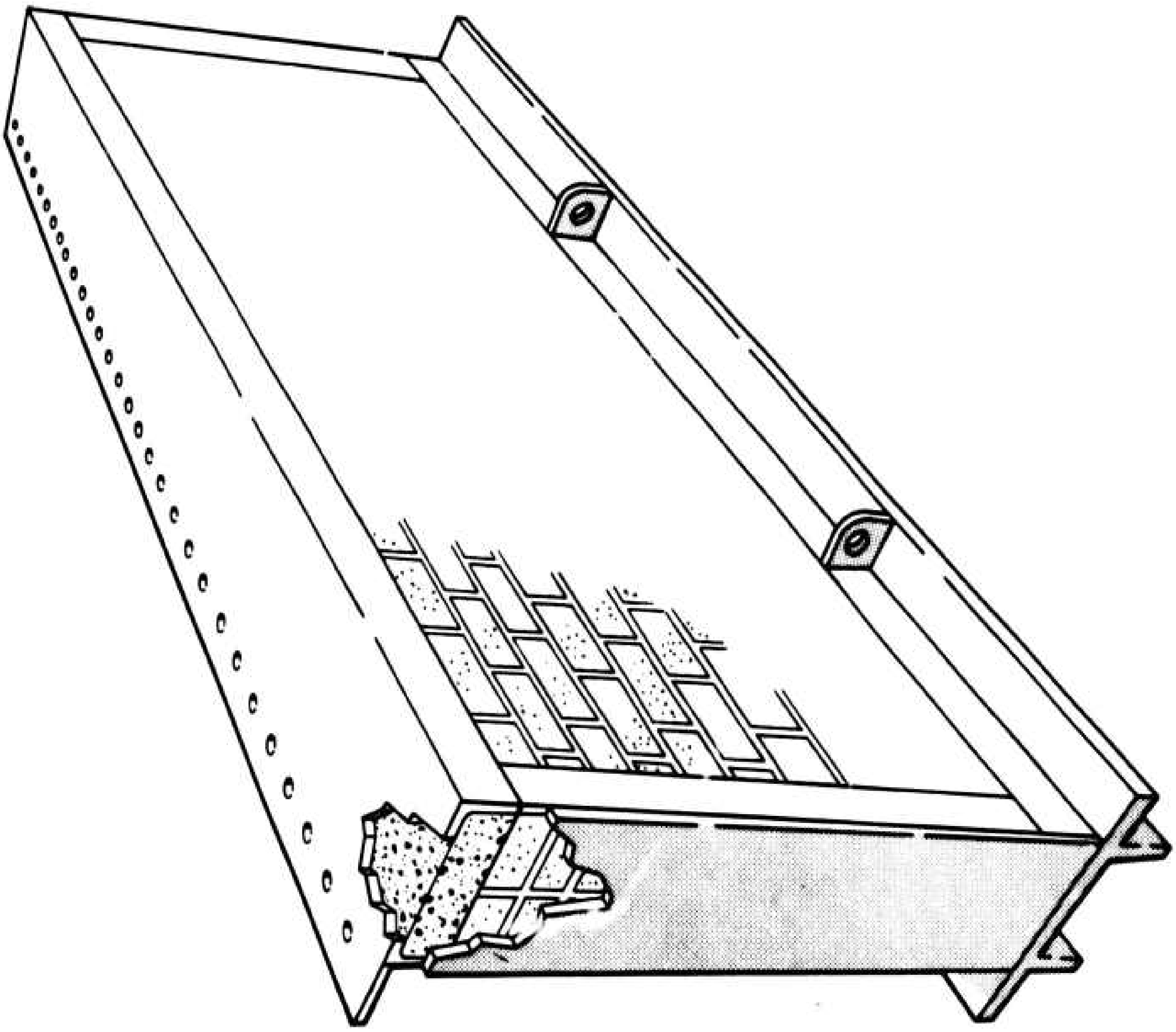


Fig. 6. Brick Panel and Frame

The brick wall panels used were non-reinforced, 8 in. thick, and were constructed of building brick and mortar. Construction detail for these panels conformed with Uniform Building Code (Ref. 4) requirements 2412-a, b, c and d. The building bricks and mortar conformed with UBC requirements 21-1-64, 24-25-64, and 24-23-64. The style of construction was "Common Flemish bond" with every sixth row a bond course.

To assure quality control in the construction of the panels and to determine the strength of the panels at the time that they are tested, a static test program has been established. At the time of construction of each panel, samples of brick and mortar are collected, and brick beams, brick columns, tensile bond test specimens and shear bond test specimens are constructed. These specimens are stored along with the panel through the 28-day curing time, and are then tested at about the same time that the panels are tested.

The static tests include the following:

- Flexure tests of individual bricks
- Compression tests of brick columns
- Flexure tests of the brick beams
- Tensile bond and shear bond tests of the brick and mortar test specimens
- Compression and splitting tests of mortar cylinders

The timber stud walls were 4-in. thick and were constructed of 3 by 4-in. Douglas fir studs on 16-in. centers and covered on both sides with 1/2-in.-thick gypsum wallboard. Construction details conformed with UBC requirements 4715 and 4716. (Ref. 4)

PANEL HARDWARE

The panel program hardware consists of the transport hardware, which includes the transporter and rollers required to hold and transport the panels from the fabrication and storage area into the tunnel, and the test hardware, which includes the wall blocks and plate girders required to hold the panels

in place and allow measurements to be made on the panels during a shock tunnel test. Each of these items will be discussed in the sequence in which they are used in the process of transporting and testing of a wall panel.

Transport Hardware

The transport hardware includes the panel transporter and the panel roller assemblies. The panel transporter is a rubber tired "tuning fork"-shaped vehicle which supports a test panel as shown in Fig. 7. Four roller chain and turnbuckle fasteners clamp onto the lifting eyes fastened to the bottom frame of the panel to be tested, and the panel is lifted by a hydraulic ram assembly. The panel transporter carries the panel from the storage area into the tunnel and places the panel on rollers. The transporter is then removed from the panel which is then rotated and positioned into place in the tunnel.

Test Hardware

The test hardware, which includes the wall blocks and plate girders, comprise a system for holding a panel in place in the tunnel and for making measurements of the total load imposed on the panel. This system is shown in Fig. 8.

The wall blocks are 3.5-in. thick steel plates 24 in. high and 30 in. long. These plates weigh about 350 lb each and are affixed to the wall by twelve 18-in. high-strength bolts per plate. The bolts are held in the concrete walls with an epoxy and each bolt is preloaded to its operating tension. This preload accomplishes two things: first, it acts as a proof load for the bolt; and secondly, with the preload at the operating level, there should never be a stress reversal in the system thus reducing the probability of brittle material fatigue. The bolts are countersunk so as not to protrude into the tunnel, and each block has a machined groove in which a key is inserted to restrain the girders during rebound in those tests where the panel has not failed.

The plate girders were designed to support the test panel along its entire length with a minimum of deflection, but still permitting rotation thus simulating a simple beam support condition. They are of rather unique welded steel construction, with "box" flanges made of structural channels and channel stiffeners. They weight about 2600 lb each and are 4-1/2 in. thick, 4 ft deep and have a span of 12 ft.

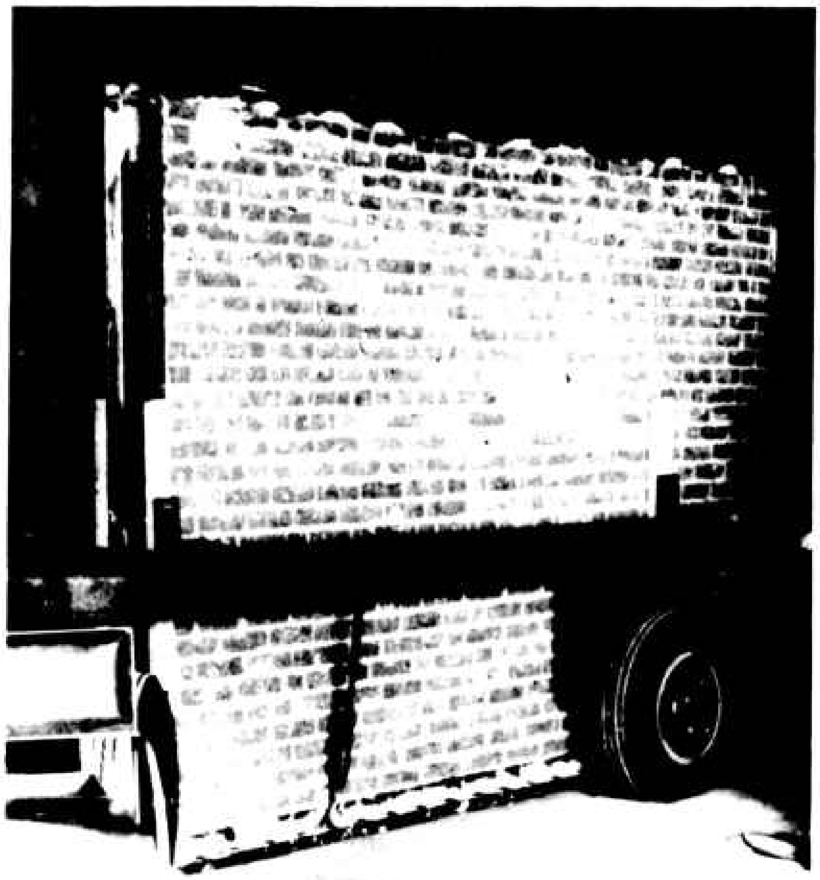


Fig. 7. Panel Transporter

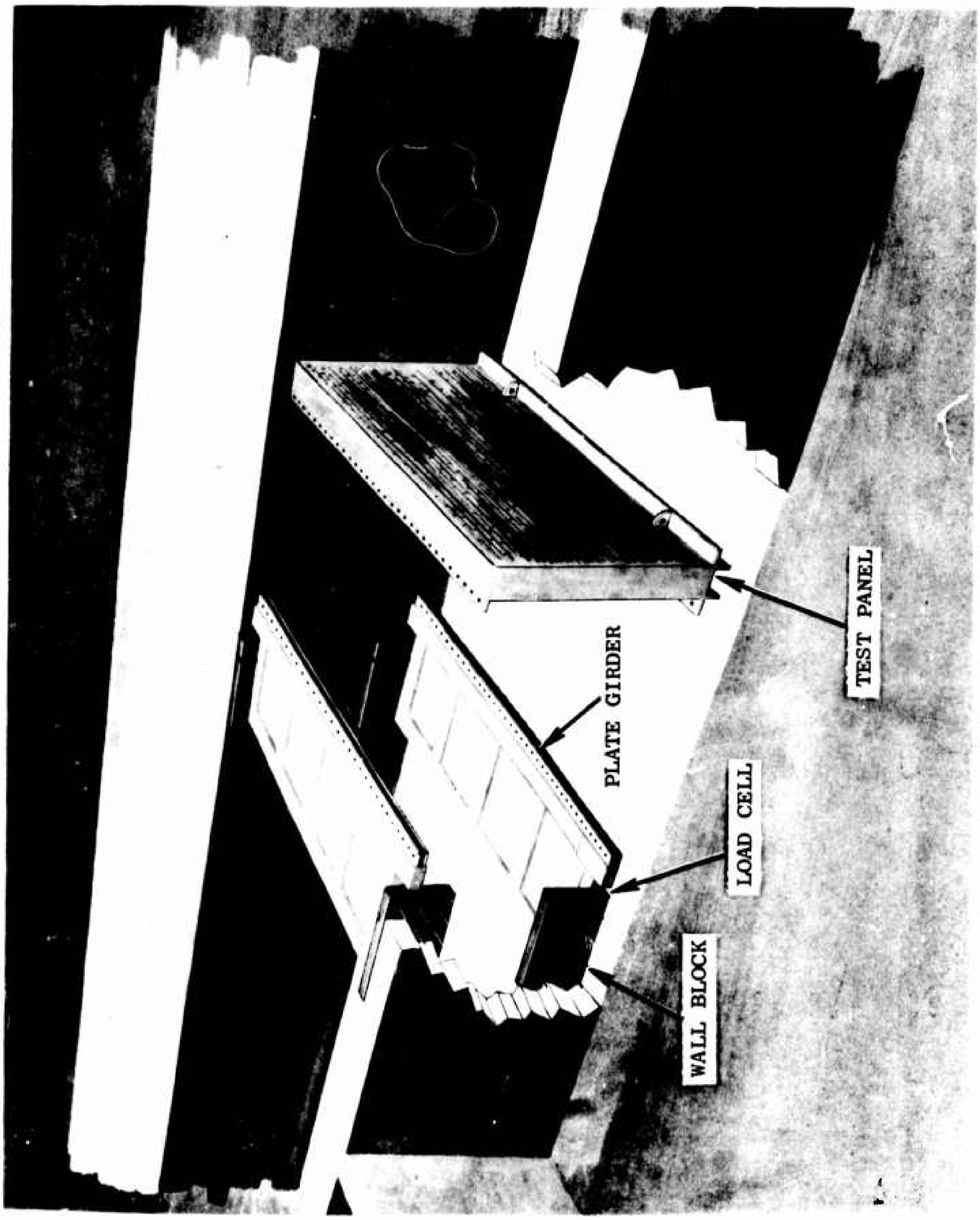


Fig. 8. Cutaway View of Shock Tunnel Showing Test Panel, and Location of Plate Girders, Wall Blocks, and Load Cells

The attachment hardware, which fastens the test panel frame to the girders, consists of a "U"-shaped member welded to the girder and a single plate attached to the top and bottom member of the test panel frame. These plates are inserted into the "U" shape and held by bolts on 6-in. centers. The holes in the plates are countersunk from both sides to allow freedom of rotation. A photograph of a panel in place in the shock tunnel is shown in Figure 9.

INSTRUMENTATION

The instrumentation for the preliminary panel test series consisted of the air pressure gauges mounted on the tunnel wall, a deflection gauge mounted in the center of the panel, load cells installed between the wall blocks and plate girders, high-speed movies and pre-shot and post-shot still pictures. The rationale behind the deflection gauge and load cell measurements is discussed in the next section.

PANEL TEST RESULTS

As noted earlier, the panel test series consisted of seven failure tests of simply supported 8-in.-thick brick panels and three failure tests of simply supported 4-in.-thick timber stud panels. Five of the brick panels and two of the timber stud panels were subjected to a peak incident pressure of ~1.5 psi, corresponding to a peak reflected pressure of ~3 psi. The remaining two brick panels and one timber stud wall were subjected to a peak incident pressure of ~ 3.5 psi, corresponding to a peak reflected pressure of ~ 8 psi.

Air Pressure Gauge Data

The shock wave characteristics in the vicinity of the panel were monitored by two gauges mounted in the tunnel wall approximately 5 ft and 13 ft in front of the test panel. The shape of the pressure pulses recorded were typically a sharp rise to the incident value, a flat step, and then another sharp rise to the peak reflected value as the reflected shock wave returned to the gauge station from the test panel. The remainder of the pulse was a nearly linear decay to zero. From these recorded pulse shapes and the results obtained in the calibration series described in Part I Section 4, it can be inferred that the pulse shape seen by the test panel was a sharp rise to the peak reflected overpressure value, a more or less flat top approximately 30 msec in duration, and

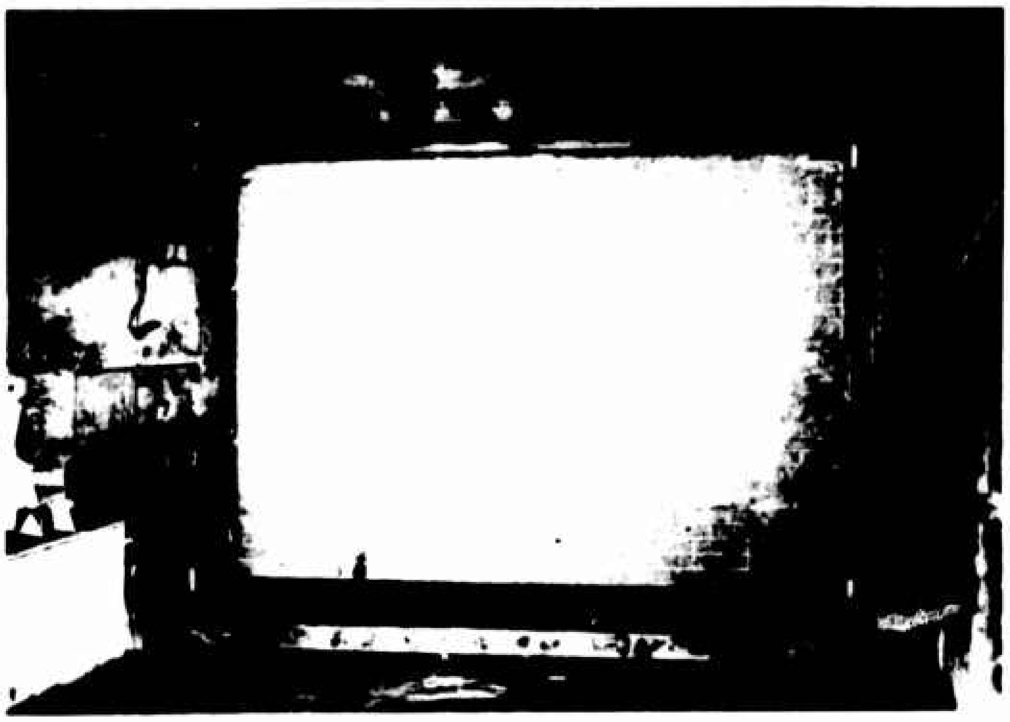


Fig. 9. Brick Test Panel in Place in Shock Tunnel

then a linear decay to zero, with an overall pulse duration of approximately 90 msec. It is possible, however, that the acceleration of the panel may have modified the tail of this pulse to some extent.

Film Coverage

High-speed films were taken of each panel test using a 16-mm Fastax camera positioned approximately 20 ft behind the test panel location. Analysis of these films shows that the overall failure process of the individual panels, for the same test condition, was remarkably similar.

In the five low-pressure tests (3 psi reflected) the characteristic failure pattern was a single, nearly horizontal break across the full width of the panel. The break was not always along a single mortar joint but usually crossed several bricks and involved two or three mortar joints across the panel. The location of the break varied somewhat but it was always in the middle third of the panel. (This variation is not surprising since the tensile stresses do not change much in this region). Subsequent to the initial break, the upper and lower sections of the panels rotated back and remained essentially intact until they impacted on the floor of the tunnel. Post-shot photos indicated that essentially all the debris from these tests was located within about 15 ft of the test location, with a few fragments out to about 50 ft. Apparently these few fragments came from the initial break zone near the center of the panel. A photo of debris from one of these tests is shown in Figure 10.

The initial failure patterns for the two higher pressure tests (approximately 8 psi reflected) were similar to those for the lower pressure tests, except that in one case two nearly parallel breaks occurred. Postshot photos from these tests indicated that the majority of the debris was located from 20 to 60 ft from the wall with only a few individual fragments left near the panel. Two photos of debris from one of these tests are shown in Figure 11.

Deflection Gauge Data

A deflection gauge was used in some tests to measure the initial deflections of the center of the panel (up to the first 0.125 in.) The gauges were mounted in a rigid frame placed in front of the panel to be tested, i.e., toward the explosion.

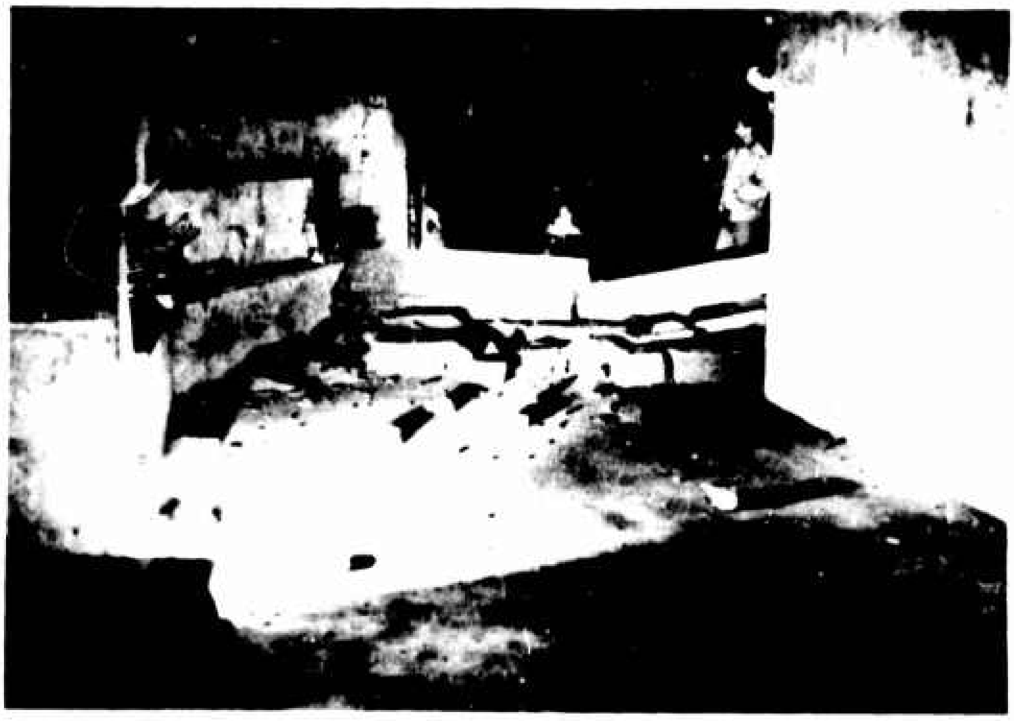


Fig. 10. Debris from a Failure Test of a Simply Supported Brick Panel (peak reflected overpressure 3 psi)

Fig. 11. Debris from a Failure Test of a Simply Supported Brick Panel (peak reflected overpressure 8 psi)

Deflecti ' gauge data from three panel tests are presented in Fig. 12, which is a plo. of deflection in inches vs time in milliseconds. Included are data from two brick panel and one timber stud wall failure tests. The peak reflected overpressures for these tests were approximately 3 psi. Note that the panels moved the first 0.125 in. in about 15 msec in the brick panel tests and in about 4 msec in the timber stud wall test.

L d Cell Data

Four 200,000-lb-capacity load cells were positioned between the wall panel-plate girder assembly and the blocks affixed to the wall of the tunnel. (See Fig. 6). In this location the load cells measure the load vs time imposed on the wall blocks during breakup of the wall panel. Typical load cell traces from the brick panel failure series are presented in Fig. 13.

Electrically the load cells and their associated recording systems are operating as expected. The calibration data obtained to date tends to confirm the factory calibration values, and in two dynamic calibration tests, in which a 12-in. -thick timber wall was placed in the test frame, the recorded load cell values received were larger by a factor of approximately 2 than the load value computed from the air pressure gauge data. This is as expected since the estimated dynamic load factor is also approximately 2.

However, as can be seen in Fig. 13 the shapes of the pulses from the panel failure tests are rather complicated. This is not too surprising because of the rather complex nature of the load transfer from the failing test panel, frame, and plate girder assembly to the load cell. Although there is no particular reason to doubt the validity of this method of total load measurement, it will be necessary to conduct a more extensive test series with a nonfailing wall at a higher range of loads than was possible with the timber wall described above.

DISCUSSION OF TEST RESULTS

It is premature to attempt to discuss the test results in any detail because of the limited nature of the testing conducted to date. However, several interesting implications of the brick panel results are worthy of note.

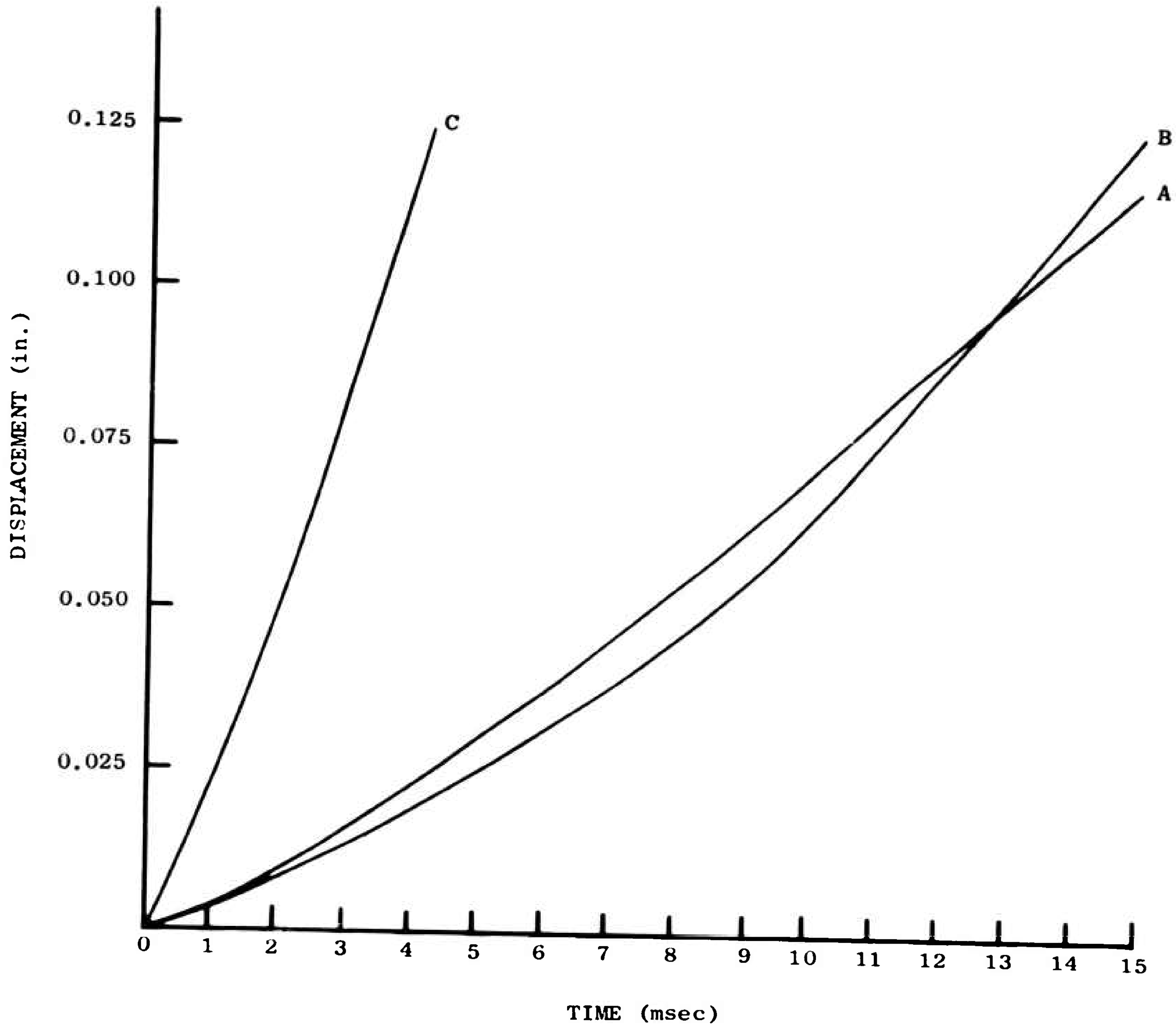
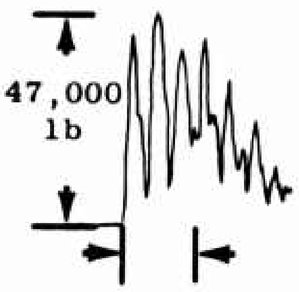


Fig. 12. Deflection Gauge Data from two Brick Panel Failure Tests (curves A and B), and one Timber Stud Wall Test (curve C). Peak reflected overpressure – 3 psi for all tests.



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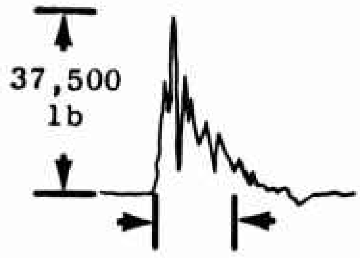

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Fig. 13. Typical Load Cell Traces from Brick Panel Failure Tests

In order to plan the initial test series, some approximate dynamic response estimates were made based on static test results of many other investigators. A modulus of elasticity of 2×10^6 psi was selected for the brick-mortar composite from a range of 1×10^6 to 5×10^6 psi and a modulus of rupture of 200 psi from a range of 40 to 400 psi. Based on these assumptions, the simple beam support condition, a square-wave loading pulse, and classical mechanics, the natural period for the panel was estimated to be 30 msec, the failure pressure to be 1 psi, and the failure time to be 15 msec. The deflection of the center of the panel at failure was estimated to be about 0.03 in. plus deflections in the support system (which are believed to be small compared to this).

Since the strength data in the literature were so scattered, a peak loading pressure of 3 psi (corresponding to an incident shock overpressure of about 1.5 psi) was selected as the initial test pressure. A range of 2:1 on deflections and $\sqrt{2}:1$ on natural period would not have been surprising.

A preliminary analysis of the static test results from some of the small brick and mortar beams (made as part of the control samples) indicated a static modulus of elasticity of about 1.5×10^6 psi and a static modulus of rupture of about 175 psi, somewhat lower values than used in the preliminary calculation. These changes would increase the natural period to about 35 msec, decrease the failure pressure to about 0.9 psi, and increase the deflection at failure to about 0.04 in. For the loading pressure of 3 psi, the failure deflection of 0.04 in. would be expected to be reached in about 7 msec.

The brick panel test results appear entirely consistent with the above rather simple analysis in that the 3-psi loading failed the wall in all cases and, from the subsequent panel motions, seemed well above the failure loading. Even more interesting is the fact that the experimentally determined deflection – time traces from the panel tests (Fig. 12) are consistent with the load – deflection – time relationships estimated above. From Fig. 12 it can be seen that the estimated failure deflection of 0.04 in. was experimentally observed to be around 7 msec, which also was the estimated value. Although this close an agreement between theory and experiment is probably fortuitous at this stage of the study, it does tend to verify that unreinforced brick panels have failure times less than about 15 to 20 msec and that the shock tunnel loading pulses are more than adequate for studying the failure mechanisms of such panels.

Section 2
DESIGN OF PANEL TEST PROGRAM

OBJECTIVES

The basic objective of the panel program will be to develop a better understanding of the mechanics of wall panel failure under blast loading, including the resultant debris characteristics. This information is needed as a basis for developing methods for predicting structural response and debris formation in the detail and with the reliability necessary for personnel casualty and property damage estimation, for debris distribution estimation, and for providing guidance in slanting new construction for protection from all weapon effects.

The approach will be to conduct an extensive experimental program on selected types of external and interior wall panels and partitions. The results of these tests combined with existing test information and theoretical work will be used as the basis for developing the improved theories of loading, response, and debris formation. The panel types studied will be selected to be representative of a class of panels whose general structural response characteristics are expected to be similar. It is anticipated that predictions for response and debris functions for other types of panels in the same class can be made on the basis of the improved theories without having to conduct additional extensive test series. Only a moderate amount of additional testing should be necessary to verify the applicability of the theory.

The program is divided into two parts: a loading study and a wall panel program. A discussion of the information requirements and general approach planned for investigating each of these areas is given in the following along with specific experimental test programs. It should be emphasized, however, that the test programs as outlined are simply indicators of the parameters likely to be of concern and of the estimated overall scope of testing required rather than firm specifications. As the program proceeds, certain parameters will likely turn out to be more important and others less important than now visualized. In addition, some unexpected phenomena, requiring further study,

will no doubt be encountered. Thus continual deletions, additions, and other modifications to the specific test design will be required during the course of the program.

LOADING STUDY

General Discussion

Panel loading information is needed for design of the panel response tests, for proper interpretation of their results, and as a check on blast wave and panel response instrumentation.

The latter requirement is of most interest for panels with no openings since for this case the panel loading can be determined from the incident shock wave characteristics. By studying the loading on panels without openings the cross-sectional uniformity of the incident shock wave can be verified and the support hardware and panel instrumentation systems can be checked out.

Loading distribution information is much more critical for panels with openings. Even with a uniform shock wave incident on the panel, the actual load distribution on the front surface is nonuniform because of rarefaction waves generated at the opening. When the opening is an appreciable portion of the total panel area, the shock wave passing through the opening is expected to provide a significant load to the back surface (which also will be nonuniform) at an early enough time to influence the failure process. The effective load distribution on the panel is thus the net difference between the front and back surface load distributions.

The third area where loading information is needed is for interior panels (with and without openings) which are loaded by the blast wave after it passes through an opening in an exterior wall. The maximum loading on the interior partition will not occur instantaneously, but rather will buildup to its maximum value in a time dependent on the area of the front panel opening and the volume of the room. If the interior panel has no opening (or if the opening is small compared to that in the exterior panel), it is expected that the pressure buildup in the room would be fast enough so that the maximum load on the interior panel for many cases of concern (long clearing time, moderate

size opening area and room volume), would be nearly the same as for a directly loaded interior panel.

One of the most interesting and realistic situations is where the interior panel also has a significant opening. Here, because of pressure relief through the opening, the maximum front surface load on the interior partition could be very much less (than that for a directly loaded panel) and in addition there could be a significant back-surface load, so that the net load on the partition could be even smaller.

The opening in the exterior wall panel may occur normally, as in the case of a window, or it may result from failure of the exterior panel. Only the former case has been included in the program planning to date since the scope of the investigation needed for the latter case will depend on information learned from the exterior panel study with regard to failure mechanisms and failure times.

Approach

The basic approach planned for the exterior wall panel loading study is to test with a modular type nonfailing exterior wall panel, instrumented with pressure gauges distributed over the front and back surfaces. The panel modules will be designed to provide stepwise variation in opening area and in opening location. This arrangement will suffice for the basic uniformity study and for the exterior wall panel loading study.

A sketch showing a preliminary design of the nonfailing wall is given in Fig. 14. In this design the wall would be composed of a frame consisting of 5 vertical and horizontal I beams and 20 modular panels. These panels would be made of 4 1/2-in. thick plywood in a steel frame and constructed so that they are easily installed, are interchangeable, and can be placed on both the front and the back of the wall. In Figure 14 the wall is shown with four of the modular panels removed, creating an opening of approximately 15 percent. (To simplify the sketch only the fastening plates for the two modular panels directly under the opening have been shown.)

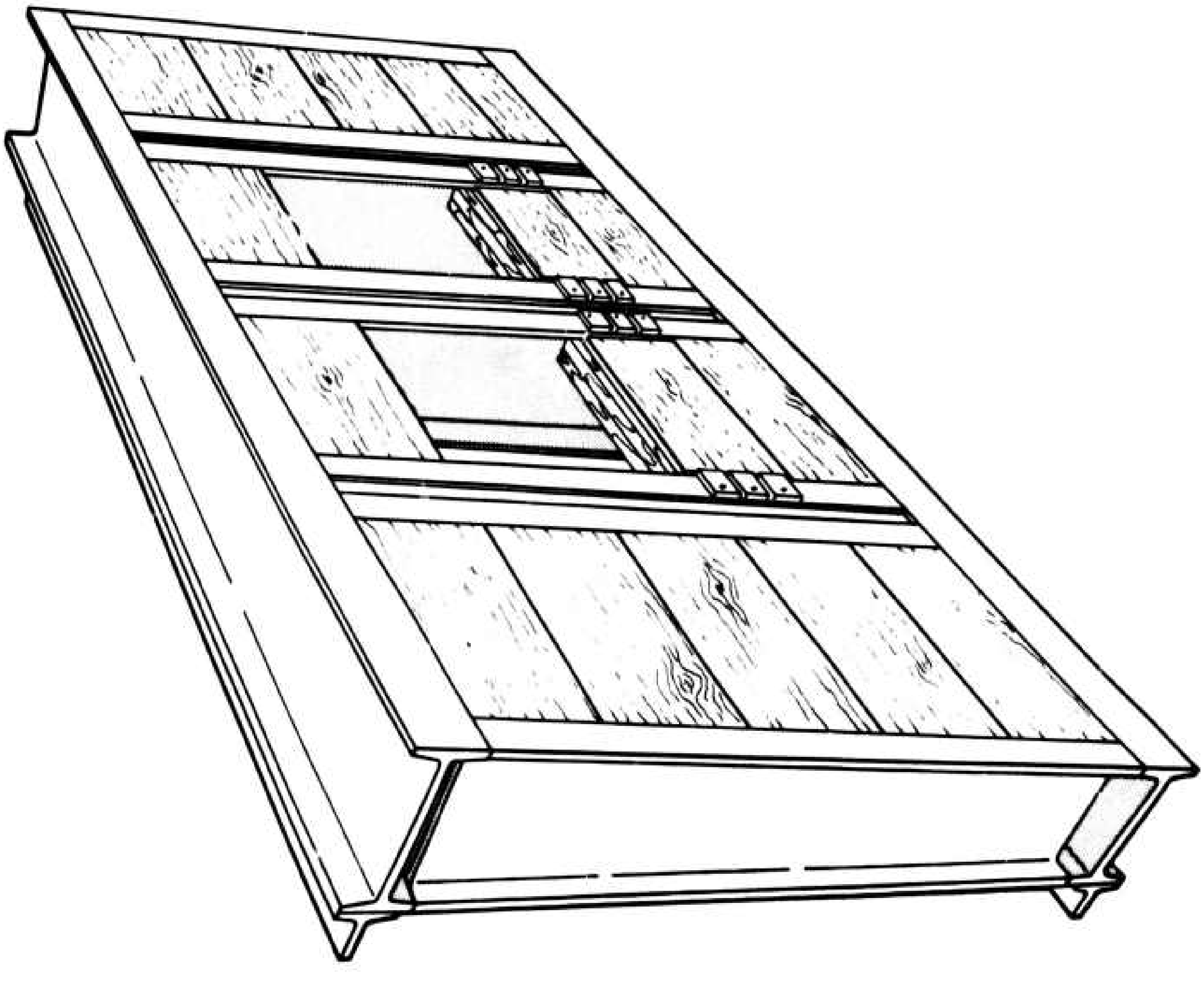

Fig. 14. Preliminary Design of Non-Failing Wall

Although the nonfailing wall design can provide larger openings, it is initially planned to limit the testing to openings having a maximum of about 30 percent of the total wall area. (Information given in Ref. 5 suggests that approximately 70 to 80 percent of the walls in NFSS structures have window openings smaller than this).

In the second phase of the loading study concerned with interior panel loading, a second modular panel, similar to the first, would be added making a room in the tunnel.

The suggested loading test series designed in accordance with the foregoing rationale is given in Table 2. The second and third colums give the size of the exterior wall panel opening (in percent of the total wall area) and its location. The specific locations, designated as 1, 2, and 3, have not been selected, but they will include a symmetrical case (opening in the middle of the wall) and a nonsymmetrical case (opening near one side). The fourth and fifth columns give similar information about the interior partitions. It is hoped that by the time interior partitions are reached, sufficient loading and response information will have been obtained that two locations for the opening will suffice. Columns 6, 7, and 8 show the range of loading conditions planned. For the early tests both pressure level and duration will be varied, while in the later tests it is assumed that at least one of these can be dropped (shown in the table as duration). The last three columns of the table show the number of repeat tests for each condition, the total number of tests for each basic panel configuration, and the cumulative total number of tests.

THE WALL PANEL TEST PROGRAM

In discussing the wall panel test program it is convenient to consider it in terms of the four major elements of the program:

1. The type or characteristics of wall panels which will be investigated.
2. The manner in which the panels will be supported.
3. The nature of the loading conditions which will be applied to the panels.
4. The measurements needed to document the loading and response characteristics of the panels.

Table 2
LOADING STUDY
NON FAILING WALLS

Case	Wall Panel % Open	Wall Panel Location	Interior Partition % Open	Interior Partition Location	Loading P_1	Loading P_2	Loading P_3	Loading T_2	Loading T_3	Number of Tests: Repeats	Number of Tests: Total	Number of Tests: Cumulative Total
0	0	-	0	-	X	X			X	2	6	6
1	30	1	None	-	X	X			X	2	6	12
2	30	2	None	-	X	X			X	2	6	18
3	30	3	None	-	X	X			X	2	6	24
4	15	1	None	-	X	X			X	2	6	30
5	15	2	None	-	X	X			X	2	6	36
6	15	3	None	-	X	X			X	2	6	42
7	30	1	15-20 (door)	1	X	X			-	2	4	46
8	15	1	15-20 (door)	1	X	X			-	2	4	50
9	30	1	15-20 (door)	2	X	X			-	2	4	54
10	15	1	15-20 (door)	2	X	X			-	2	4	58
11	30	2	15-20 (door)	1	X	X			-	2	4	62
12	15	2	15-20 (door)	1	X	X			-	2	4	66
13	30	2	15-20 (door)	2	X	X			-	2	4	70
14	15	2	15-20 (door)	2	X	X			-	2	4	74

Wall Panel Characteristics

The selection of the types and characteristics of wall panels to be included in the test program was based largely on a consideration of the types of wall panels which are sufficiently common to be of concern on a nationwide basis and on a consideration of the differences in response anticipated between the various types of panels.

It is recognized that some previcus experimental work has been conducted which is applicable to wall panel behavior. This includes, for example, wall panel tests at the Nevada Test Site (Ref. 6), large-scale floor slab tests conducted by the Waterways Experiment Station (Ref. 7), small-scale wall panel tests at the Illinois Institute of Technology Research Institute (Ref. 8), and other static and dynamic tests such as those conducted at the Unit-Masonry Association.

The range of support and loading conditions investigated in these programs, however, particularly for those concerned with full-scale (or large-scale) panels, was quite limited. Thus, although the available information will contribute to the development of response theories and may well reduce the amount of testing needed for some panel types, it will not reduce the basic classes which should be studied.

From a material response point of view a reasonable initial classification of panel types would be as follows:

<u>Non-Reinforced Masonry</u>

Brick
Tile
Concrete block

<u>Reinforced Concrete and Masonry</u>

Concrete
Brick
Concrete block, etc.

<u>Lightweight Construction</u>

Timber stud - various coverings
Steel stud - various coverings, etc.

Nonreinforced masonry panels are expected to exhibit basically brittle types of failure with essentially elastic responses up to failure. Thus, a linear theory seems appropriate. Because of the brittle nature of the material, a fair amount of statistical scatter in material properties is expected.

Reinforced concrete and masonry panels are expected to exhibit a fairly broad spectrum of responses, from near-ductile for reinforced concrete to near-brittle for reinforced brick. For this group of materials less statistical scatter is expected, but the theory will probably be nonlinear in nature.

Lightweight construction covers a fairly wide range of wall panels and interior partition types so that a variety of responses is expected. For example, the frame of a timber stud wall is expected to show a ductile behavior, while many of the wall coverings, such as sheetrock, are brittle materials. One characteristic of this group is that connection failures are expected to be fairly common because of the general weakness in the connections.

Within this general framework, the remaining guidance on the selection of panel types to be included in the test program was obtained from existing information on the frequency of occurrance of various panel types. The most applicable such information located to date was the RTI survey of structural characteristics of NFSS structures in the five cities (Ref. 5). From the survey data the frequency of occurrence of various types of exterior walls and interior partitions was determined. Table 3 gives the basic frequency data for exterior wall panels and Table 4 the data for interior partitions. From these tables it can be seen that brick is the most common exterior wall type and concrete is second. For interior partitions, timber stud walls are most common, followed by structural tile and nonreinforced concrete block.

The panel frequency data are summarized in Table 5 in terms of the panel classification system discussed above. It can be seen that nonreinforced brick panels are the logical selection for the typical panel type to be studied in the nonreinforced masonry category since they have by far the largest frequency of occurrence. A similar line of reasoning leads to the selection of reinforced concrete and timber stud walls as the characteristic panels to study in the other two panel categories.

Table 3

NFSS EXTERIOR WALL PANEL SURVEY DATA

	Panel Type	Fraction of Walls	Cumulative Fraction of Walls
1.	Nonreinforced Brick - Bearing Wall	0.23	
2.	Cast in Place Concrete - Bearing Wall	0.15	0.38
3.	Nonreinforced Brick - Curtain Wall	0.11	0.49
4.	Tile - Curtain Wall - Masonry Veneer	0.08	0.57
5.	Nonreinforced Concrete Block - Curtain Wall	0.06	0.63
6.	Cast in Place Concrete - Curtain Wall	0.04	0.67
7.	Nonreinforced Concrete Block - Bearing Wall	0.04	0.71
8.	Nonreinforced Brick - Bearing Wall - Masonry Veneer	0.04	0.75
9.	Nonreinforced Concrete Block - Curtain Wall - Masonry Veneer	0.03	0.78

OTHER WALL TYPES IN ORDER OF OCCURRENCE

10. Nonreinforced Brick - Curtain Wall - Masonry Veneer
11. Cast in Place Concrete - Bearing Wall - Masonry Veneer
12. Nonreinforced Concrete Block - Bearing Wall
13. Tile - Curtain Wall
14. Reinforced Concrete Block - Bearing Wall - Masonry Veneer
15. Studwall - Timber - Bearing Wall
16. Stone - Bearing Wall
17. Cast in Place Concrete - Curtain Wall - Masonry Veneer
18. Reinforced Brick - Bearing Wall
19. Studwall - Timber - Curtain Wall - Masonry Veneer
20. Metal Panel - Curtain Wall
21. Precast Concrete - Bearing Wall - Masonry Veneer
22. Reinforced Concrete Block - Bearing Wall
23. Structural Tile - Bearing Wall - Masonry Veneer
24. Precast Concrete - Curtain Wall
25. Reinforced Concrete Block - Curtain Wall - Masonry Veneer
26. Reinforced Brick - Bearing Wall - Masonry Veneer
27. Stone - Bearing Wall - Masonry Veneer
28. Precast Concrete - Curtain Wall - Masonry Veneer
29. Precast Concrete - Bearing Wall
30. Reinforced Brick - Curtain Wall
31. Studwall - Timber - Curtain Wall
32. Metal Panel - Curtain Wall - Masonry Veneer

Table 4

NFSS INTERIOR PARTITION DATA

Partition Type	Fraction of Partitions*	Cumulative Fraction of Partitions
1. Timber Studwall - Non-bearing	0.24	
2. Structural Tile - Curtain	0.17	0.41
3. Nonreinforced Concrete Block Curtain	0.15	0.56
4. Nonreinforced Brick - Bearing Wall	0.10	0.66
5. Cast in Place Concrete	0.07	0.73

OTHER PARTITION TYPES IN ORDER OF OCCURRENCE

6. Movable Partitions - Non-bearing
7. Nonreinforced Concrete Block - Load Bearing
8. Nonreinforced Brick - Non-bearing
9. Gypsum Block - Non-bearing
10. Reinforced Concrete Block - Non-bearing
11. Cast in Place Concrete - Non-bearing
12. Timber Studwall - Load Bearing
13. Tile - Load Bearing
14. Precast Concrete - Load Bearing
15. Reinforced Brick - Load Bearing
16. Reinforced Concrete Block - Load Bearing

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* 0.17 of the buildings had no partitions.

Table 5

SUMMARY OF NFSS EXTERIOR WALL PANEL AND INTERIOR PARTITION DATA

General Class	Type	Exterior		Interior	
		Fraction of Total	Cumulative Fraction	Fraction of Total	Cumulative Fraction
Nonreinforced Masonry	I. Nonreinforced Brick with/without veneer	0.38	0.38	0.10	0.10
	II. Nonreinforced Concrete Block with/without veneer	0.13	0.51	0.15	0.25
	III. Structural Tile	0.08	0.59	0.17	0.42
Reinforced Concrete and Masonry	IV. Reinforced Concrete	0.19	0.78	0.07	0.49
Light Weight Construction	V. Timber Stud	-	-	0.24	0.73

The results in Table 5 also suggest that the highest priority in the testing schedule should be given to the brick panels since the nonreinforced masonry category accounts for about 50 percent of all panels, both exterior and interior. Timber stud walls appear appropriate as the second priority because they are the most common interior partition and because their ease of construction and installation will permit their testing to be rather easily scheduled between the brick panel tests.

Once the basic types of panels have been selected, there are still numerous other panel variables which need to be considered. These include for example: the percent of essentially open area in the panel (windows or doors), the quality of the materials used in the construction (reflected in their strength and elastic properties), and variations in construction practice and material sizes. Clearly an attempt to study all of these variations in the test program would be prohibitive from a cost point of view. It also is not technically justifiable at present since there is good reason to believe that the effects of many of these variables can be adequately predicted from the response theories developed.

For these reasons the philosophy adopted has been to investigate in the initial basic test program only those variables which existing knowledge suggests will have significant effects on the mechanism of response.

On this basis the only panel variable selected for detailed study is the percent of open area (and its location). This variable is not only of interest from a response point of view, but also because it can have a marked effect on the panel loading conditions. As discussed previously in connection with the loading study (Section 1) panel openings in the range from 0 to 30 percent of the total panel area are considered of most interest. The basic panel response will be studied first with no openings and then with values of 15 to 30 percent, with at least two different locations of the openings, symmetrical and non-symmetrical.

The remaining panel variables will be held constant throughout the majority of the test program and the only variations planned will be to verify that the effects of these variables are as predicted by the panel response theories

developed from the basic program. In order to control these other variables in a sound manner it is planned to run a rather complete static test program parallel to the dynamic test program. In the static test program it will be possible to make sufficient tests to insure that we have indeed "controlled" variables omitted from the basic program. It is currently the intent to run mortar control tests in compression and tension (splitting) and to run brick quality control tests in flexure (modulus of rupture). It is also intended to run composite control tests to determine the mortar – brick rupture modulus (brick beam tests), mortar – brick tensile bond, and mortar – brick shear bond. A sufficient number of tests are planned to permit evaluation of the random variations found in the tested materials. This will provide a rational basis of control so that one can indeed say wall group 1 is the "same" (statistically) as wall group 2, etc. In addition, these tests will provide a start on the static test program, which will provide the static-to-dynamic link needed to predict dynamic response of a wall panel from the proper static test data.

Panel Support Conditions

Because of the variety of panel support conditions that exist, some generalization is necessary to bring the number of test conditions within manageable bounds. For this purpose the relatively classical support conditions listed below seemed most appropriate since they individually or in combination bound all real cases:

1. Simply Supported. By simple support it is meant that the support condition permits no deflection in the direction of the loading (normal to the plate), freedom to rotate about the support, and no restraint in the plane of plate.

2. Fixed or Continuous Support. The fixed or continuous support condition permits no deflection in the direction of the load, no rotation about the support and no restraint in the plane of the plate.

3. Preloaded Support. The preloaded support means a superposition of a load in the plane of the plate, traditionally imposed by using the plate (wall) as a load-bearing member. However, these preloading stresses can be induced by shrinkage, drying, temperature changes, and so forth, and hence, can occur in directions other than the gravity direction and can be in both directions in real structures. In the test program, preload will be confined to one direction to study its effect. In addition, the preloaded support condition can occur with either a simple support condition or the fixed support condition. The allowable bearing stresses in bearing walls are quite low, therefore elastic behavior is still expected to prevail in the brittle materials test program.

4. Restrained Support Condition. The restrained support condition occurs when wall panels are confined within and/or between massive frame members. Therefore the term "restrained" means that motion in the plane of the plate is restricted. In this case it is possible to develop a nonelastic behavior and achieve very high stress levels, e.g., the ultimate strength of the brick–mortar combination in compression.

Although a somewhat idealized approach has been used, each of these generalized conditions does simulate an actual condition and is not just an academic case. These support conditions are generally applicable to the panel acting as a "beam" (supported on two opposite sides), and as a "plate" (supported on all four sides).

Figure 15 illustrated the types of support conditions and the proposed sequence in which they will be investigated. These are:

Type 1 - Simple Beams. The simple beam is the easiest to analyze and test, hence a logical starting point. However, the selection of the simply supported wall panels is not just based on operational ease but it also fits into the "real" world. Many one-story brick walls are placed on bearing foundations which offer little or no moment resistance at the base and are held at the top by a light roof diaphragm, such as plywood. This type of construction is very close to a simple support condition. Even in buildings where there are vertical supports, such as walls, if the height of the wall is less than one-half the distance between supports, the supports have little effect in the center. In addition, this type of test will yield the moment resistance of a brick panel remote from a support of any type.

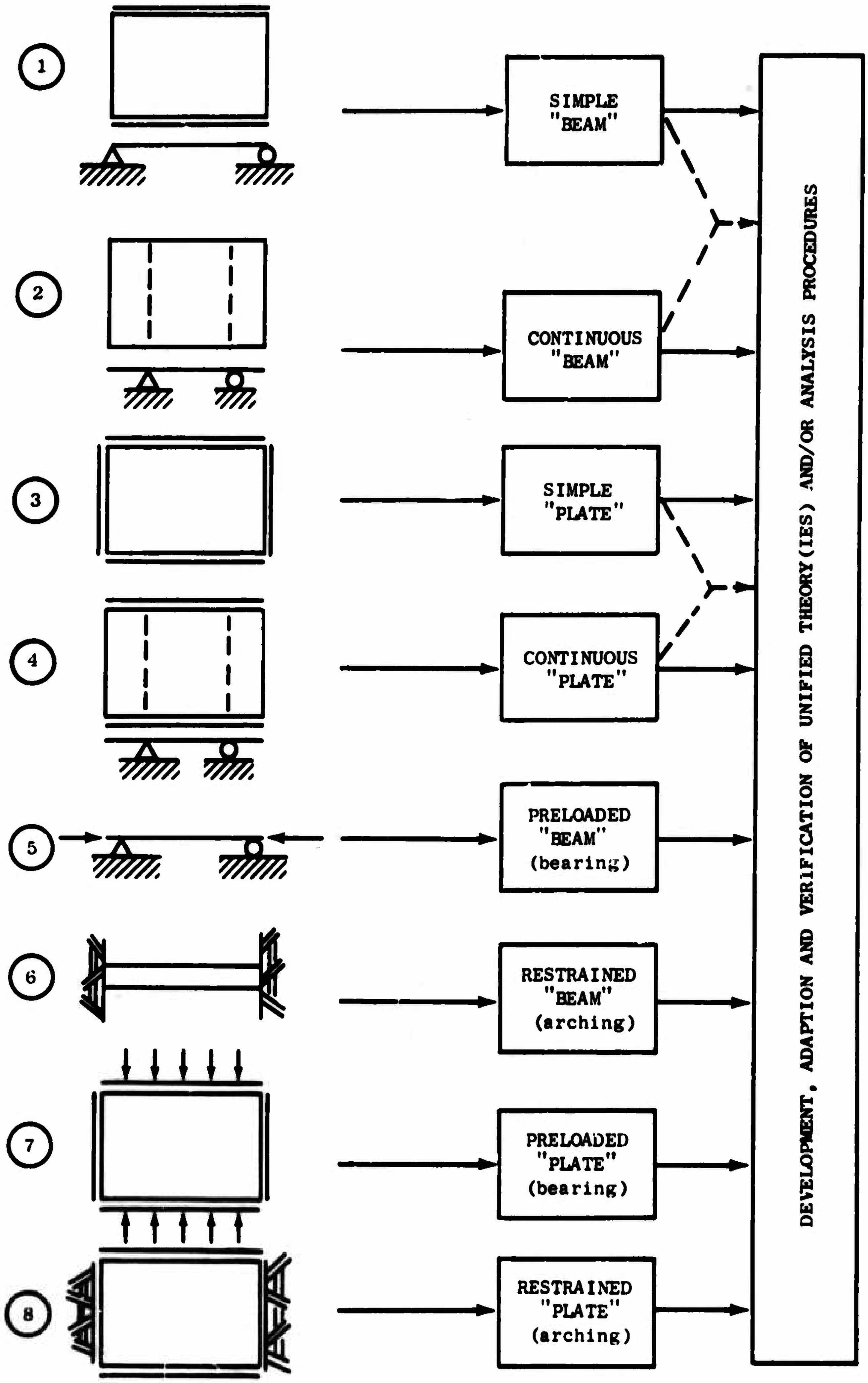


Fig. 15. Support Conditions

Type 2 - Continuous Beam. The continuous beam support will supply information on the moment capacity at a fixed support or a continuous support. The continuous support is selected in preference to a fixed support because it is easier to construct and verify its behavior. Fixed supports are very difficult to build for they imply infinite support rigidity; hence there is always the question on the degree of fixity. This type of test directly models a wall on a moment-resistant foundation and/or a parapet above the roof, a top story with parapet etc. In addition, the knowledge obtained here plus that from the simple beam support condition (Type 1), allows us to combine continuous, fixed, and simple support conditions for many additional configurations.

Type 3 - Simply Supported Plate. The simply supported plate will supply the information needed to add the two dimensional effects of vertical support to the tests of Type 1 support, such as walls and columns. One of the major problems to plague experimentors in testing simply supported plates is the meaning of "simply supported," i.e., no deflection at the support, but free to rotate. This requires the design of test fixtures to "hold down" the corners as they tend to pop up and change the plate strength.

Type 4 - Continuous Plate. This test series would extend the tests of Type 2 into the two-dimensional domain of vertical supports in the building with fixed foundations, parapet walls, etc. At this point in the program, it should be possible to analyze from a support point of view panels in most one- and two-story buildings plus the upper stories of many multistory buildings with non bearing walls.

Type 5 - Preloaded Beam. This test series is designed to add in the effects of vertical loads on bearing walls of Types 1 and 2. This would extend the analysis capability to multistory structure and structures where the wall actually supports the roof and/or floors.

Type 6 - Restrained Beam. This kind of support occurs in massive rigid frame type of structures, where the panel has complete axial restraint. This axial restraint changes the panel behavior from that of a flexural type of member to an arching or membrane type of member.

Type 7 - Preloaded Plate. This support condition extends the effort and knowledge of Type 5 to the two-dimensional situation.

Type 8 - Restrained Plate. The restrained plate extends Type 6 to the two-dimensional case.

Loading Conditions

The parameters of the loading pulse which are of concern are the peak value, the duration, and the pulse shape.

The basic rationale used to specify the range of peak values to be used in the testing for any given panel is relatively straightforward once the panel type, pulse shape, and pulse duration are specified. In general, the lower limit to the level of interest is that value which is just sufficient to fail the panel (designated the failure pressure level). Since the failure mechanism, debris generation characteristics, and loading transferred to the frame during the failure process may well change with peak value, there is also concern with peak values above the failure level. Accordingly, in the testing it is planned, in general, to use the failure pressure (or slightly higher) and at least one significantly higher pressure level (and in some cases two significantly higher values). The specific pressure levels will be selected during the course of the program, but it is anticipated that there will be at least a factor of two between each level.

The selection of the pulse shapes and durations to be used in the testing is somewhat more difficult since the range of loading conditions of interest on wall panels is quite large, even for weapons in the megaton range. Consider, for example, the loading on a panel on the front face of a building oriented parallel to the shock front. At the instant of shock contact with the building, the loading corresponds to the peak reflected pressure in the shock wave. Pressure relief waves then start moving in from the edges of the building, causing the pressures to decrease below the peak reflected value and to ultimately reach the stagnation pressure value. For the average loading on the entire front face, the time for the pressure to decay to the stagnation value is given approximately by $t = 3S/U$, where S is the half-width or height of the front face and U is the shock front velocity. For a panel near the edge of the building, the decay will start almost immediately after initial reflection, while for a panel near the center of the ground floor, the decay will not start until the first relief wave arrives, at a time of approximately S/U. For the range of pressures of most interest (up to about 12 psi) there is roughly a factor of two difference

between peak reflected pressures and stagnation pressures. Thus, even though pressures in the incident blast wave may not have changed appreciably during several passage times of the shock over the building, the actual loads on panels can change appreciably.

Since panel failure times, which are expected to be in the tens of milliseconds, are very much less than the total duration of the incident blast wave, most concern with regard to the panel failure mechanism is in the shape of the loading pulse during the first 50 to 100 msec. One limit for this pulse shape is for locations near the center of the front face of large buildings, where S is of the order of 40 or 50 ft. In this case, the loading would be essentially constant during the first 30-40 msec, and then would decay during the next 60-80 msec to the stagnation value. At the other limit for small buildings, where S is, say, 10 ft, and for locations near the edge of the building, the loading would start to decay immediately and stagnation values would be reached in about 20 msec.

Of the various pulse shapes which can be encountered in real situations it is believed that the general type of most interest in the test program would be similar to the first one described above, i.e., a pulse which has an initial flat top lasting for, say, 30 to 40 msec, followed by a decay to stagnation pressure by about 100 msec. This is partly because the initial flat-top shape is easiest to treat from a theoretical point of view, partly because of the great interest in NFSS structures, which are typically rather large buildings, and partly because with the present total duration limit in the shock tunnel facility, this pulse shape will provide the largest total impulse for a given peak loading.

It is also believed that variations in the pulse shape of the type described above are not very likely to have a significant effect on the failure mechanism and thus there is reason to believe that their effect can be reasonably well predicted once the failure process is understood for one generalized pulse shape condition.

Accordingly, in the test plan design the majority of the testing has been allocated to the selected pulse shape, although a limited number of tests with other pulse shape conditions (or duration conditions) will be conducted to verify the ability to predict such effects.

The shock tunnel results (described in Part I Section 4) indicate that pulses can be obtained using the type of explosive charge arrangements tested to date, which simulate the desired pulse for the first 80 to 100 msec. Thus, little additional development work will be necessary to provide the loading conditions needed for investigating panel failure mechanisms.

Two different exposure conditions are planned for studying interior panels, both using the basic pulse shape described above. In one case the panel will be exposed directly to the loading pulse and in the other case it will be mounted behind the partially open nonfailing wall used for the loading study. The initial tests will be made using direct loading on the assumption that understanding of the failure mechanism for a simple pulse shape is necessary before investigating the more complicated pulse shapes obtained for the other cases. The relative emphasis that will be put on the two cases in the overall program, however, will not be decided until some information has been obtained from the loading study and from the initial direct loading tests.

In the discussion so far consideration has been limited to the pulse shape and duration requirements for studying the panel failure mechanisms. This, however, is not the only objective of the panel tests. In addition it is desired to investigate the characteristics of the debris resulting from the panel failure and the loads transferred to the frame during the process. A complete simulation of the full-scale load durations in the shock tunnel with regard to these aspects, particularly debris, does not seem practical since this would require total pulse durations in excess of several seconds. Although, with further development, it is anticipated that some increase in pulse duration can be obtained in the tunnel, it seems unlikely, that, durations of more than a few hundred msec will be obtained.

Since the stresses induced in the panel due to the initial load will have been largely relieved once the panel has failed, it is expected that little further breakup of the panel fragments will occur until these fragments hit the ground or other walls. Thus beyond the panel failure time, major interest is in the rigid body motion of the panel fragments under the blast flow conditions as modified by any restraints still imposed at the edges of the panel due to the supports. This problem appears amenable to an analytical approach since at least to a first approximation it does not involve nonelastic structural response.

The planned approach is to determine the particle size and velocity distributions at the end of the loading duration. This information will then be used as initial conditions for the calculation of the motions of the fragments due to the subsequent blast flow.

To verify the results of these calculations, it is planned to conduct a limited number of tests with a loading pulse having a significantly different duration (and impulse). Preferably longer durations than the standard pulse will be used, but this will depend on further improvements in the shock tunnel loading system.

Measurement System

The measurement requirements for the test program can be conveniently divided into four categories:

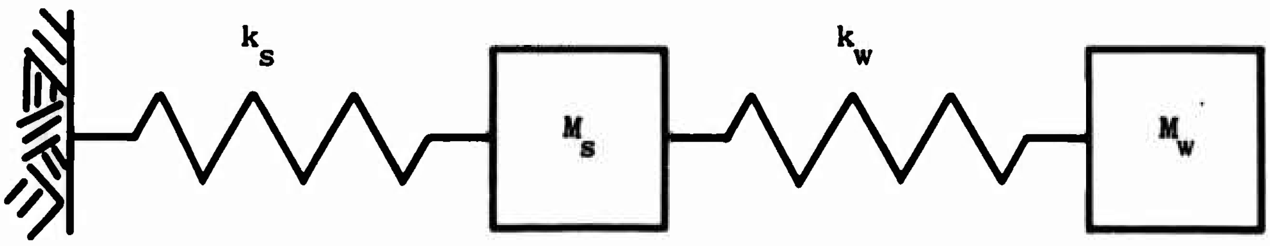
1. Those needed to document the blast wave characteristics incident on the panel.
2. Those needed to measure the load distribution on the panels.
3. Those needed to measure the load transferred from the panel to the frame.
4. Those needed to measure the panel response.

It is anticipated that the input blast wave characteristics (overpressure vs time) can be adequately documented by means of pressure gauges mounted in the walls of the tunnel in the same general manner as used in the tunnel tests conducted to date (see Section 1). The only changes anticipated are modifications in the mounting or relocation of the gauge positions to reduce spurious acceleration signals.

It is also planned to use pressure gauges mounted in the nonfailing wall panel to measure the load distribution on the panels. It is recognized that there may be problems associated with this approach because of the difficulties in making the nonfailing panel wall rigid enough to reduce spurious acceleration signals in the gauges to an acceptable level. It may be necessary, for example, to use rather elaborate mounting systems for the gauges, such as having them independently mounted and protruding through the walls rather than having them supported by the wall.

Although ultimately it may be desirable to measure all components of load transferred to the frame from the panel, in the initial stages it is planned to limit the measurements to the component normal to the panel face. This load component will be measured by a system similar to the basic load cell/plate girder assembly used in the preliminary test series. In this arrangement the panel load was transfered to two rigid plate girder assemblies, which in turn applied the load to four load cells mounted on the wall blocks.

Since the plate girder support assembly (as well as any other supporting system) will have a mass which is a significant portion of even a heavy panel, such as brick, there is some problem in interpreting the load cell data. It will be necessary to measure the frequency response of the elastic system, including the panel and the support structure. It is hoped that a sufficiently accurate mathematical model can be evolved using a two-lump-mass system as shown below:



Here k_s and M_s represent the support structure stiffness and mass, respectively, and k_w, M_w represent the stiffness and mass of the wall, respectively. The quantities k_s, M_s, k_w and M_w will be determined mathematically and experimentally. It is then intended to vibrate the system and determine the resonant (primary) frequencies of the system. In this specific design the support structure is designed to be very stiff such that the transmitted load is essentially <u>only</u> the panel response. However, with the more complete mathematical system the panel response, the system response and the support structure response can be predicted for less tailored designs.

During the failure process both k_w and M_w undergo rapid change as far as the system response is concerned. By knowing the earlier elastic response of the system and the continued elastic response of the support structure, the nonelastic failure process of the failing wall panel may be deduced.

The measurement requirements for documenting the panel response are probably the most difficult to specify. Clearly, it would be sufficient if the stresses, strains, and deflections at every point in the panel were measured as a function of item. This however, is impractical and no doubt unnecessary. The optimum arrangement, however, cannot be specified at present since it depends in part on the actual panel failure mechanisms and on how well certain types of instrumentation perform in the tunnel. Thus, the measurement system described below should be considered only as a starting point for the optimum system, which will evolve over a period of time as additions and changes are made.

For the panel response measurements it is planned initially to use high-speed photography, deflection gauges, and strain gauges. Two high-speed cameras are planned, one located at the open mouth of the tunnel viewing the back side of the panel (the position used in the tests conducted to date (see Section 1), and the other mounted in the side of the tunnel about 8 ft behind the panel. This camera will be used to provide a close-up view of the panel during the failure process and also in an endeavor to measure the initial debris velocity and size distributions.

During the preliminary series of tests, some photographic problems were encountered due to difficulties in adequately lighting the tunnel and to obscuration due to dust. To minimize these problems, it is planned to paint the tunnel wall with a light-colored sealer, which should reduce dust generation and improve the overall light levels. Providing that the majority of the dust is not coming from the panel breakup itself, these measures should permit reasonably good photographic data to be obtained.

On the wall itself, the intent is to measure both the deflection and the strain vs time. Deflection will be measured by an expanded version of the system checked out in the preliminary test series, which used a linear variable differential transformer as the sensor. Because of the composition of brick walls, i.e., constructed of discrete elements, some development of measuring techniques will be necessary to get good strain readings. This will be checked by the one-to-one relationship between strain and deflection. By combining the deflection and strain knowledge with the load cell and pressure transducer

data, one will be able to deduce the fiber stresses and the effective moduli of elasticity.

A comparable set of data will be taken on static tests, that is, stress, strain, deflection, and elastic modulus numbers. These will then be compared to the dynamic results of the panel tests. The intent here is two-fold; first the static tests will offer a measure of quality control and sample consistency and, secondly, it is hope to evolve a set of static tests that will be of value in dynamic performance prediction.

Recommended Program

Tables 6, 7, and 8 summarize the recommended programs for the brick panels, timber stud walls, and reinforced concrete panels, respectively. The first column gives the panel type; the second, the panel thickness, for brick and concrete, and the panel facing material for the timber study wall. The third column gives the panel opening (in percent of the total panel area) and location of the opening (designated as I or II). The specific location conditions will be selected after the loading study has been completed. The fourth through eleventh columns indicate the panel support conditions that will be used, and the twelfth through fifteenth columns, the loading conditions. The last three columns give the number of repeat tests for each condition, the total number of tests for each basic case, and the cumulative number of tests.

In addition to the tests listed in these tables it is also recommended that some tests be conducted specifically for the purpose of evaluating slanting techniques as they are suggested by the results of the basic program.

Table 6

BRICK PANEL PROGRAM

Panel Type			Mounting*								Loading				Number of Tests		
			Beam				Plate					2nd	3rd	2nd			Cumulative
Material	Thickness	Opening	S	C	P	R	S	C	P	R	Failure	Pressure	Pressure	Time	Repeats	Total	Total
1. Brick	8	0	X								X	X	X	X	3	12	12
2. Brick	8	0		X							X	X		X	2	6	18
3. Brick	8	0					X				X	X	X	X	2	8	26
4. Brick	8	0						X			X	X		X	2	6	32
5. Brick	8	0			X						X	X			2	4	36
6. Brick	8	0				X					X	X	X	X	2	8	44
7. Brick	8	0							X		X	X			2	4	48
8. Brick	8	0								X	X	X			2	4	52
9. Brick	X	0					?				X				2	2	54
10. Brick	X	0				?					X				2	2	56
11. Brick	8	30% I					X				X		X		2	4	60
12. Brick	8	30% II	?					?			X		X		2	4	64
13. Brick	8	15% I					X				X		X		2	4	68
14. Brick	8	15% II	?					?			X		X		2	4	72

* S-Simple, C-Continuous, P-Preload, R-Restrained

Table 7

TIMBER STUD WALL PROGRAM

PANEL TYPE			MOUNTING*								LOADING				NUMBER OF TESTS		
MATERIAL	FACING	OPENING	BEAM				PLATE				FAILURE	2ND PRESSURE	3RD PRESSURE	2ND TIME	REPEATS	TOTAL	CUMULATIVE TOTAL
			S	C	P	R	S	C	P	R							
1. T.S. Wall	gypsum board	0	X								X	X	X	X	2.5	10	10
2. T.S. Wall		0					X				X		X		2	4	14
3. T.S. Wall	metal-lath & plaster	0	X								X		X		2	4	18
4. T.S. Wall	metal-lath & plaster	0					X				X		X		2	4	22
5. T.S. Wall	plywood	0	X								X		X		2	4	26
6. T.S. Wall	plywood	0					X				X		X		2	4	30
7. T.S. Wall	X	15-20% (door)	X								X		X		2	4	34

Some of the tests in this series will be repeated for both general loading conditions of concern; directly exposed to the incident shock wave, and indirectly exposed behind a non-failing exterior panel with an aperture. The relative emphasis that will be placed on the two loading conditions will be established on the basis of the results from the loading study and the initial directly loaded wall tests.

* S-Simple, C-Continuous, P-Preload, R-Restrained

Table 8

REINFORCED CONCRETE PANEL PROGRAM

	Panel Type			Mounting*								Loading				Number of Tests		
				Beam				Plate					2nd	3rd	2nd			Cumulative
	Material	Thickness	Opening	S	C	P	R	S	C	P	R	Failure	Pressure	Pressure	Time	Repeats	Total	Total
1.	Concrete	8 in.	0		X							X		X		3	6	6
2.	Concrete	8 in.	0						X			X	X	X	X	2	8	14
3.	Concrete	8 in.	0			X						X		X		2	4	18
4.	Concrete	8 in.	0							X		X		X		2	4	22
5.	Concrete	8 in.	0				X					X		X		2	4	26
6.	Concrete	8 in.	0								X	X	X	X	X	2	8	34
7.	Concrete	8 in.	30%	X								X		X		2	4	38
8.	Concrete	8 in.	15%	X								X		X		2	4	42
9.	Concrete	8 in.	30%						X			X	X	X	X	2	8	50
10.	Concrete	8 in.	15%						X			X		X		2	4	54

* S-Simple, C-Continuous, P-Preload, R-Restrained

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UNCLASSIFIED
Security Classification

DOCUMENT CONTROL DATA - R&D

(Security classification of title, body of abstract and indexing annotation must be entered when the overall report is classified)

1. ORIGINATING ACTIVITY (Corporate author) URS Corporation 1811 Trousdale Drive Burlingame, California	2a. REPORT SECURITY CLASSIFICATION UNCLASSIFIED 2b. GROUP

3. REPORT TITLE
Development and Evaluation of a Shock Tunnel Facility for Conducting Full-Scale Tests of Loading, Response, and Debris Characteristics of Structural Elements

4. DESCRIPTIVE NOTES (Type of report and inclusive dates)

5. AUTHOR(S) (Last name, first name, initial)
Willoughby, A. B.
Wilton, C.
Gabrielsen, B

6. REPORT DATE	7a. TOTAL NO. OF PAGES	7b. NO. OF REFS
December 1967	79	8

8a. CONTRACT OR GRANT NO. Subcontract Nos. 11229(6300A-320) b. PROJECT NO. 11618(6300A-250) c. d. OCD Work Units 3313B, 1123D	9a. ORIGINATOR'S REPORT NUMBER(S) URS 680-2 9b. OTHER REPORT NO(S) (Any other numbers that may be assigned this report)

10. AVAILABILITY/LIMITATION NOTICES
This document has been approved for public release and sale; its distribution is unlimited.

11. SUPPLEMENTARY NOTES	12. SPONSORING MILITARY ACTIVITY Office of Civil Defense Office of the Secretary of the Army Washington, D. C. 20310

13. ABSTRACT

This report describes the conversion of an existing large-cross-sectional-area tunnel into a shock tube and the evaluation of the capabilities of this facility for blast loading and response studies of full-scale and large-scale structural elements. Also included are the results from a preliminary test series of full-scale wall panels conducted as part of the evaluation program and the design of a detailed test program to investigate the loading, response, and debris characteristics of wall panels.

The basic tunnel, which is a section of a former coastal defense complex, has a 8-1/2- by 12-ft rectangular cross section and is 163 ft long.

Shock waves are generated in the expansion chamber of the tunnel by detonating strands of primacord which have been uniformly distributed throughout a section of the compression chamber.

Evaluation tests conducted to date indicate that a wide range of air blast conditions can be generated depending on the explosive arrangement. Both peaked and flat-topped pulse shapes have been obtained, with total durations approaching 100 msec and flat-topped durations of about 40 msec.

The preliminary wall panel test series, which included seven 8-in.-thick nonreinforced brick panels and three timber stud walls, confirmed the suitability of the shock tunnel for full-scale panel testing. The failure process was reproducible, and the failure times, even for brick panels, were much less than the loading durations.

DD FORM 1 JAN 64 1473

UNCLASSIFIED
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14 KEY WORDS	LINK A		LINK B		LINK C	
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